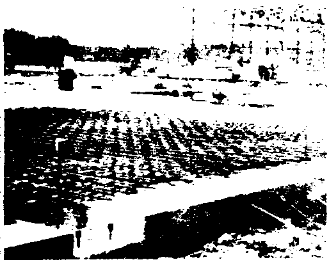
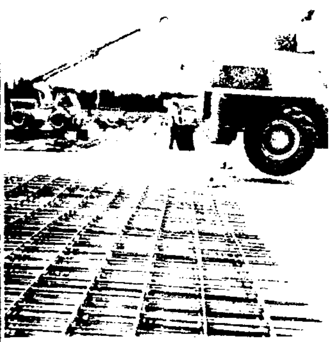
US Army Corps of Engineers





GEOTECHNICAL LABORATORY

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(2)

PROCEEDINGS OF THE WORKSHOP ON DESIGN CONSTRUCTION, AND RESEARCH FOR RIBBED MAT FOUNDATIONS

by

Lawrence D. Johnson

Geotechnical Laboratory

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US ARMY ENGINEER WATERWAYS EXPERIMENT STATION

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Final Report

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19. ABSTRACT (Continue on reverse if necessary and identify by block number)

This report presents the minutes of the proceedings of the workshop conducted on the design, construction, and research for ribbed mat foundations conducted at the US Army Engineer Waterways Experiment Station 25 to 27 August 1987. The purpose of the workshop was to examine concepts and philosophy for research in mat foundations to determine the direction of future work. This purpose was accomplished through four sessions: Design/Construction Methodology in Practice, Construction and Performance, Active Research, and Proposed Research Plan.

The results of the workshop indicated that an important priority of future work should be constructing, monitoring, and analyzing data from field test sections to understand better the performance of foundations in expansive soil. The effect of climate is an important aspect of this study. In preparation for field test sections, the following work should be

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Expansive soil
Foundation construction
Foundation design
Heave
Mat foundations
Mat repair
Ribbed mats
Soil-structure interaction
Swelling soil

19. ABSTRACT (Continued).

accomplished: (1) a field survey of Corps of Engineers Division and District offices, real estate developers, contractor organizations, casualty insurance writers, consultants, and educational institutions to collect a detailed list of all design/construction procedures, local practices, and their usefulness/effect on performance, (2) development of a systematic damage record system and requirement for repairs, (3) a field survey to measure surface displacements with the systematic performance record system, (4) pursue development of ground modification techniques that reduce potential volume changes leading to design/construction of more economical foundation systems, and (5) pursue development of a frequency spectrum model that provides guidelines of acceptable movements for a given foundation system.

Field test sections will demonstrate the above developments and assist evaluation of soil input parameters such as edge moisture variation distance, active depth of heave, maximum and minimum soil wetting profiles, and effectiveness of ground modification techniques. Field tests section studies should be a cooperative effort between interested agencies and organizations.

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PREFACE

The minutes of the workshop proceedings, "Design, Construction, and Research for Ribbed Mat Foundations," were prepared for the Office, Chief of Engineers, US Army, under RDT&E Work Unit AT22/AO/010, Mat Foundations for Intermediate and Heavy Military Structures.

This workshop was organized under the direction of a steering committee for mat foundation research. Members of this steering committee were Professor W. Kent Wray, Department of Civil Engineering, Texas Tech University, Lubbock, Texas; Professor G. Wayne Clough, Department of Civil Engineering, Virginia Tech, Blacksburg, Virginia; Mr. Al Branch, Jr., Foundation and Materials Branch, Fort Worth District, Corps of Engineers (CE); Mr. Bill H. James, Southwestern Division (SWD), CE; Dr. Lawrence D. Johnson, Research Group, Soil Mechanics Division (SMD), Geotechnical Laboratory (GL), Waterways Experiment Station (WES); and Mr. Clifford L. McAnear, Chief, SMD, GL, who was also the advisor for the committee. Mr. Joseph P. Hartman, SWD, participated in the steering committee in the absence of Mr. James. The proposed research plan and topics presented at the workshop were prepared by Professor Wray under an Intergovernmental Personnel Agreement from 1 June 1987 to 31 August 1987. The workshop was coordinated and minutes prepared by Dr. Johnson under the supervision of Mr. McAnear and Dr. William F. Marcuson III, Chief, GL. Many participants contributed to the preparation of these minutes, particularly Mr. Robert Crisp, Consultant, Marietta, Georgia; Mr. R. Gordon McKeen, Consultant, Albuquerque, New Mexico; Mr. William R. Stroman, Consultant, Fort Worth, Texas; and Mr. Robert Yunker, Pacific Ocean Division, CE.

COL Dwayne G. Lee, CE, was the Commander and Director of WES at the time of the workshop. Dr. Robert W. Whalin was Technical Director.

CONTENTS

CONVERSION FACTORS, NON-SI TO SI (METRIC)
UNITS OF MEASUREMENT

Non-SI units of measurements used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
acres	4,046.873	square metres
acre-feet	1,233.481	cubic metres
cubic yards	0.7645549	cubic metres
Fahrenheit degrees	5/9	Celsius degrees or Kelvins*
feet	0.3048	metres
feet per second	0.3048	metres per second
gallon (US liquid)	3.785412	cubic decimetres
gallons per minute	3.785412	cubic decimetres per minute
horsepower (550 ft-lb per sec)	745.699	watts
inches	25.4	millimetres
inches per second	25.4	millimetres per second
miles (US statute)	1.609347	kilometres
pounds (force) per inch	175.1268	newtons per metre
pounds (force) per square foot	47.88026	pascals
pounds (force) per square inch	6894.757	pascals
pounds (mass)	0.4535924	kilograms
square feet	0.9290304	square metres
square yards	0.8361274	square metres
tons (2,000 lb, mass)	0.9144	kilograms

* To obtain Celsius (C) temperature readings from Fahrenheit readings, use the following formula: $C = (5/9)(F - 32)$. To obtain Kelvin (K) readings, use: $K = (5/9)(F-32) + 273.15$.

PROCEEDINGS OF THE WORKSHOP ON DESIGN, CONSTRUCTION, AND RESEARCH FOR RIBBED MAT FOUNDATIONS

INTRODUCTION

1. The workshop, "Design, Construction, and Research for Ribbed Mat Foundations was held at the US Army Engineer Waterways Experiment Station (WES) on 25-27 August 1987. This workshop was sponsored by RDT&E Work Unit AT22/AO/010 entitled "Mat Foundations for Intermediate and Heavy Military Structures."

2. The purpose of this workshop was to examine concepts and philosophy for research in mat foundations, particularly for mats in expansive soil areas applicable to facilities designed and constructed by the Corps of Engineers. Research conducted in the United States tends to be fragmentary with no clear coordination of complementary research efforts or technology transfer between various organizations and agencies. The scope includes discussions on alternative foundation types and repairs where performance or site characteristics influences the selection, design and construction of mat foundations.

3. The purpose of the workshop was accomplished through the four sessions of the agenda shown in Table 1 concerning Design/Construction Methodology in Practice, Construction and Performance, Active Research, and Proposed Research Plan. The workshop was organized through a steering committee consisting of Professor W. Kent Wray (Texas Tech University), Joseph P. Hartman (Southwestern Division or SWD), Al Branch, Jr. (Fort Worth District or FWD), Professor G. Wayne Clough (Virginia Tech), and Lawrence D. Johnson (WES). A list of the attendance is provided in Table 2.

Table 1

Agenda

Tuesday, Aug 25, 1987	Subject	Speaker
0830-0845	Welcome	COL D.G. Lee W. F. Marcuson III
0845-0900	Workshop Objectives	L. D. Johnson (WES)
	SESSION I: DESIGN/CONSTRUCTION METHODOLOGY IN PRACTICE	
0900-0930	Overview of Design and Construction	W.K.Wray (Texas Tech University)
0930-1015	PTI Design Procedure	W. K. Wray
1015-1030	Break	
1030-1115	SWD Design Procedure	J.P.Hartman (SWD)
	SESSION II: CONSTRUCTION AND PERFORMANCE	
1115-1145	Construction Constraints and Concerns	Jack Fletcher (SWD)
1145-1245	FHA Experiences	D. Earl Jones (Consulting Engineer)
1245-1330	Lunch	
1330-1415	Repair of Facilities	W. R. Stroman (Consulting Engineer)
1415-1430	Break	
1430-1530	Repair of Mat Foundations	M. Prager
1530-1600	Review of Handbook for Building Foundation Control During Construction	R. L. Crisp (Consulting Engineer)
Wednesday, Aug. 26, 1987	SESSION III: ACTIVE RESEARCH	
0830-1000	Contributions from Academic Community	W. K. Wray R. G. McKeen (NMERI)
1000-1015	Break	
1015-1130	Contributions from Corps	
	Southwestern Division	J. P. Hartman
	Fort Worth District	A. L. Branch (FWD)
	RDT&E Research	L. D. Johnson
	CASE	Chris Merrill (WES)

(Continued)

Table 1 (Concluded)

Wednesday, Aug. 26, 1987	Subject	Speaker
1130-1200	Problems in Need of Research	W. K. Wray
1200-1300	Lunch	
	SESSION IV: PROPOSED RESEARCH PLAN	
1300-1430	Historical Perspectives and Future Directions	R. L. Lytton (Texas A&M University)
1430-1445	Break	
1445-1515	Presentation of Basic Plan	W. K. Wray
1515-1630	Working Groups for Development of Plan	Participants
1630-1700	Brief summaries from Working Groups	Participants
Thursday, Aug. 27, 1987		
0830-1030	Instrumentation	G. W. Clough (Virginia Tech) K. Hilmer (Univ- ersity of Nurnburg)
1030-1045	Break	
1045-1115	Field Demonstration Concepts	A. L. Branch
1115-1200	Summary	W. C. Sherman, Participants

Table 2

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(Continued)

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Texas Tech University
Lubbock, TX 79409

Professor Robert. L. Lytton
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Texas A&M University
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SESSION I: DESIGN/CONSTRUCTION METHODOLOGY IN PRACTICE

Overview

4. Professor W. Kent Wray began this session with a description of 16 design procedures of which the Post Tensioning Institute (PTI) and Southwestern Division procedures appear to be used most frequently. The 16 design procedures are referenced in Table 3. The consequences of construction quality control deficiencies such as insufficient slab thickness, poor placement of reinforcement, improper post-tensioning procedures, and improper placement of column footings were reviewed. Damages to mats are often repaired by underpinning, permajacking, mudjacking, mini-piles, and epoxy crack repair.

PTI Design Procedure

5. Professor Wray reviewed this procedure which is fully documented in the report, "Design and Construction of Post-Tensioned Slabs-On-Ground", published by the Post-Tensioning Institute, 301 W. Osborn, Suite 3500, Phoenix, AZ. The procedure is based on results of parametric analysis using a plate on an elastic foundation finite element program SLAB2. Two modes of deformation, edge and center heaves, were applied in the analyses. The analyses indicated that maximum bending moments are near the edge of the mat. Complex mats should be divided into rectangular sections for design; cross-beams should be continuous throughout the mat. All input data for the design, which must be either calculated or measured, consist of structural and soil parameters:

a. Structural. Slab length and width, beam width and depth, beam spacing, magnitude of loads.

b. Soil. Allowable soil bearing pressure, edge moisture variation distance, differential soil movement, and slab-subgrade friction coefficient. Climatic conditions are considered in the edge moisture variation distance and differential soil movement values.

The design is accomplished through selection and analysis of trial sections.

SWD Design Procedure

6. Mr. Hartman reviewed this procedure which is based on the beam on Winkler foundation and fully documented in the report, "Development of Design Formulas For Ribbed Mat Foundations in Expansive Soils", US Army Corps of Engineers, Southwestern Division, Dallas, TX (Appendix A). This procedure is

Table 3

Most Frequently Used Design Procedures

a. Procedure

Procedure	Input Parameters	References (Table 3b)
Rigby & Dekena	perimeter wall load slab dimensions soil coefficient "K"	1
Salas & Serratosa	slab weight slab dimensions ultimate bearing capacity of soil swelling pressure on foundation Rigby and Dekena's "K" coefficient	2
Dawson	climate rating Cw from table USCS soil classification plasticity index minimum support area index of slab, c	3
B.R.A.B.	support area index, c climatic rating Cw modulus of elasticity of concrete plasticity index superstructure load slab dimensions	4
City of Knox (Australia)	total line load on slab effective linear shrinkages stiffening beam depth	5
Lytton	perimeter wall load interior wall load uniformly distributed loads support area index, c modulus of elasticity of concrete swelling mound exponent, m	6,7,8,9,10,11
Walsh	loading edge penetration distance modulus of subgrade reaction support area index, c maximum differential soil movement slab dimensions	12
Fraser and Wardle	computer program	13

(Continued)

Table 3a. (Continued)

Procedure	Input Parameters	References (Table 3b)
Wire Reinforcement Institute	subgrade modulus of reaction forklift truck data stack loading aisle width concrete flexural strength comcrete compressive strength concrete modulus of elasticity factor of safety	14
Panak	same as Wire Reinforcement Institute	15
PCA	subgrade modulus of reaction forklift truck data number of load repetitions concrete flexure strength concrete compressive strength maximum post loading post contact area post spacing maximum stack loading storage load layout aisle width	16
PTI	perimeter wall loads slab dimensions maximum differential soil movement edge moisture variation distance Thornthwaite moisture index permissible deflection ratio prestressing data depth to constant suction constant suction value clay content predominant clay mineral plasticity index gross soil permeability cation exchange capacity slab-subgrade friction coefficient	17
Swinburne	concrete tensile strength concrete compressive strength permissible deflection ratio slab dimensions edge moisture variation distance maximum differential soil movement	18,19

(Continued)

Table 3a. (Concluded)

Procedure	Input Parameters	References (Table 3b)
Gunalan	concrete compressive strength slab dimensions stack loading post loading forklift truck data modulus of elasticity of soil aisle width	20
Ringo-Corps of Engineers	concrete compressive strength forklift truck data modulus of subgrade reaction modulus of elasticity of concrete	21
SWD-Corps of Engineers	subgrade modulus limiting swell pressure edge moisture variation distance magnitude of total heave load magnitude: perimeter, interior slab length slab width slab cross-section reinforcement schedule	22 (Appendix A) 23 (Appendix A)

b. Design Procedure References

Number	References
1	Rigby, C. A. and Dekena, D. J., 1951. "Crack Resistant Housing," presented at the 30th Annual Conference, British Institution of Municipal Engineers, South African District.
2	Salas, J. A. J. and Serratosa, J. M. 1957. "Foundations on Expansive Clays," Proceedings, 4th International Conference on Soil Mechanics and Foundation Engineering, Vol 1, London, England, pp 424-428.
3	Dawson, R. F. 1959. "Modern Practices Used in the Design of Foundations for Structures on Expansive Soils," Quarterly of the Colorado School of Mines, Vol 54, pp 67-88.
4	Building Research Advisory Board. 1968. "National Research Council Criteria for Selection and Design of Residential Slabs-on-Ground," Publication No. 1571, National Academy of Sciences-National Research Council, Washington, D.C.

(Continued)

Table 3b. (Continued)

Number	Reference
5	Washusen, J. A. 1977. "The Behavior of Experimental Raft Slabs on Expansive Clay Soils in the Melbourne Area," Master's Thesis Presented to Victoria Institute of Colleges, at Hawthorn, Victoria, Australia, in Partial Fulfillment of the Requirements for the Degree of Master of Engineering (civil).
6	Lytton, Robert L. 1970. "Analysis for Design of Foundations on Expansive Clay," Symposium on Soils and Earth Structures in Arid Climates, The Institute of Civil Engineers, Australia, Paper No. 2872, pp 21-28.
7	Lytton, Robert L. 1970. "Design Criteria for Residential Slabs and Grillage Rafts on Reactive Clay," Report for the Australian Commonwealth Scientific and Industrial Research Organization, Division of Applied Geomechanics, Melbourne, Australia.
8	Lytton, Robert L. 1971. "Risk Design of Stiffened Mats on Clay," Proceedings of the 1st International Conference on Applications of Statistics and Probability to Soil and Structural Engineering, Hong Kong, pp 154-171.
9	Lytton, Robert L. 1972. "Design Methods for Concrete Mats on Unstable Soils," 3rd Inter-American Conference on Materials Technology, Rio de Janiero, Brazil, pp 171, 177.
10	Lytton, Robert L. 1973. "Stiffened Mat Design Considering Viscoelasticity, Geometry, and Site Conditions," Proceedings, 3rd International Conference on Expansive Soils, Vol 2, Israel Society of Soil Mechanics and Foundation Engineering, Haifa, Israel, pp 189-193.
11	Lytton, Robert L. and Woodburn, J. A. 1973. "Design and Performance of Mat Foundations on Expansive Clay," Proceedings, 3rd International Conference on Expansive Soils, Vol 1, Israel Society of Soil Mechanics and Foundation Engineering, Haifa, Israel, pp 301-307.
12	Walsh, P. F. 1974. "The Design of Residential Slabs-on-Ground," Division of Building Research Technical Paper No. 5, Commonwealth Scientific and Industrial Research Organization, Highett, Victoria, Australia.
13	Fraser, B. E. and Wardle, L. J. 1975. "The Analysis of Stiffened Raft Foundations on Ex[pansive Soil," Symposium on Recent Developments of the Analysis of Soil Behaviour and their Application to Geotechnical Structures, University of New South Wales, Kensington, New South Wales, Australia, pp 89-98.

(Continued)

Table 3b. (Concluded)

Number	Reference
14	Wire Reinforcement Institute 1975. "Design Procedure for Industrial slabs Reinforced with Welded Wire Fabric," Interim, Report, Wire Reinforcement Institute, McLean, VA, 138 pp.
15	Panak, J. J. and Rauhut, J. B. 1975. "Behavior and Design of Industrial Slabs on Grade," American Concrete Institute Journal, Vol 72, pp 219-224.
16	Packard, R. G. 1976. "Slab Thickness Design for Industrial Concrete Floors on Grade," Portland Cement Association, Skokie, IL, 16 pp.
17	Post-Tensioning Institute (PTI) 1980. "Design and Construction of Post-Tensioned Slabs-on-Ground," Post-Tensioning Institute, Phoenix, AZ, 89 pp.
18	Holland, J. E., Pitt, W. G., Lawrance, C. E. and Cimino, D. J. 1980. "The Behaviour and Design of Housing Slabs on Expansive Clays," Proceedings, 4th International Conference on Expansive Clays, Vol I, Denver, CO, pp 448-468.
19	Wray, W. K. 1980. "Discussion of The Behaviour and Design of Housing Slabs on Expansive Clays," Proceedings, 4th International Conference on Expansive Clays, Vol II, pp 757-763.
20	Gunalan, K. N. 1986. "Analysis of Industrial Floor Slabs-on-Ground for Design Purposes," Dissertation presented to Texas Tech University at Lubbock, TX in partial fulfillment of the requirements for the degree of Doctor of Philosophy.
21	Ringo, B. C. 1978. "Design, Construction, and Performance of Slabs-on-Grade for an Industry," American Concrete Institute Journal, Vol 75, pp 594-602.
22	Hartman, J. P. 1986. "Development of Design Formulas for Ribbed Mat Foundations in Expansive Soils," Unpublished Report, US Army Corps of Engineers, Southwestern Division, Dallas, TX, 27 pp (see Appendix A).
23	Hartman, J. P. and James, B. H. 1986. "Design of Ribbed Mat Foundations," Unpublished Report, US Army Corps of Engineers, Southwestern Division, Dallas, TX, 29 pp .

more conservative and easier to apply than the PTI procedure. Design equations were developed from results of parametric analyses assuming a cantilever beam for center lift and simple beam for edge lift. Guidelines are available in this report for selection of structural and soil parameters. Some restrictions are necessary; for example, maximum edge lift should normally not exceed 1 inch. The PTI procedure is also permitted with limitations such as size, loads, and differential movements.

SESSION II: CONSTRUCTION AND PERFORMANCE

Construction Constraints and Concerns

7. Mr. Jack Fletcher noted that heave can be reasonably well estimated, but the edge moisture variation distance is still elusive. Full scale instrumented mats should be investigated further. Construction problems include:

   a. Insufficient widths for ribs (i.e., 8 inches), which make steel placement nearly impossible. Minimum width should be 12 inches for constructability.

   b. Holding trenches open for placement of reinforcement and concrete.

   c. Cleaning of construction joints at top of ribs prior to placement of the slab is necessary, but often not done.

   d. Excessive concrete slumps; slumps should be 2 to 4 inches. Crushed rock is preferred rather than washed gravel.

   e. Fills must be nonexpansive with some cohesion; limits must be specified for the plasticity index.

Problems with cracks that occur in mats are related with the degree of user perception and function of the facility. Formal guidance on dealing with construction problems is not yet available.

8. Mr. William Stroman, consultant, indicated that substandard performance of many mat foundations can be traced to inappropriate construction techniques. Many construction people do not understand that ribs and slabs are designed to act in concert; the slab-rib system is not given proper respect during construction. Corps of Engineer inspectors should be given training to improve this situation. Mr. R. Gordon McKeen, New Mexico Engineering Research Institute, indicated that these construction problems may need to be considered separately from research requirements.

FHA Experiences

9. Mr. D. Earl Jones, recently retired from the Federal Housing Administration, provided options for soil treatment and construction in different field situations. The site may be treated to be compatible with the structure or the structure designed to accommodate or resist soil movements. Options for site modification were provided in an unpublished paper, "Options for Building on Expansive Clays", to participants of the workshop. The BRAB design procedure (Reference 4, Table 3b) results in ribbed mats that will resist

differential movement, but these mats cost $3,000 to 4,000 extra per house. The PTI procedure provides a flexible slab, which requires a compatible superstructure of sufficient flexibility for good performance. The PTI procedure works if mats are properly constructed and post-tensioned. A chart, Figure 1, indicates damages experienced in residential structures.

10. Mr. Jones estimated annual damages of about 10 billion dollars, which occur mostly in pavements. These damages occur slowly and are usually not newsworthy, except in isolated incidents where accumulated heave leads to catastrophic failure. For example, gas lines had ruptured a few years ago in a school building located in east Texas due to differential movement in heaving soil. The leaking gas led to an explosion in the building killing about 300 school children, but the cause of the accident was not attributed to expansive soil at the time.

11. Mr. Jones discussed several problems that occur in measuring swell pressure and properly forcasting location and extent of swell. Different methods of measuring swell pressure provide different answers. Greater compaction increases swell pressure; 100 percent modified compaction also gives 10 to 20 times more swell potential than 80 percent compaction. Strength data can be superimposed on compaction curves to determine soil strength for particular swell, density, and water content distributions.

12. Mr. Jones concluded with some innovative construction techniques and soil treatments. Construction on hillsides should be on overcuts and fill; do not build on a cut and fill site. Greased PVC sleeves on drilled shafts can reduce skin friction and uplift thrust, but moisture may migrate further down the shaft than without sleeves and no advantage is gained. Deep benchmarks must be sealed. A chemical stabilization process using potassium ions in a proprietary catalyst (Soil Technology Corporation) increases soil permeability and destroys the swell potential. Mr. McKeen indicated that the composition of the stabilizer and method of adding the stabilizer to the soil depends on the soil; thorough soil testing is needed before using at a particular site. Past treatments required injection down to about 85 percent of the active zone and cost about $3.50/sq. ft. More study is needed in the area of soil and site characterization to determine effects of time rates of heave, depth of active zone, limits of seasonal moisture variations, and edge moisture variation distance.

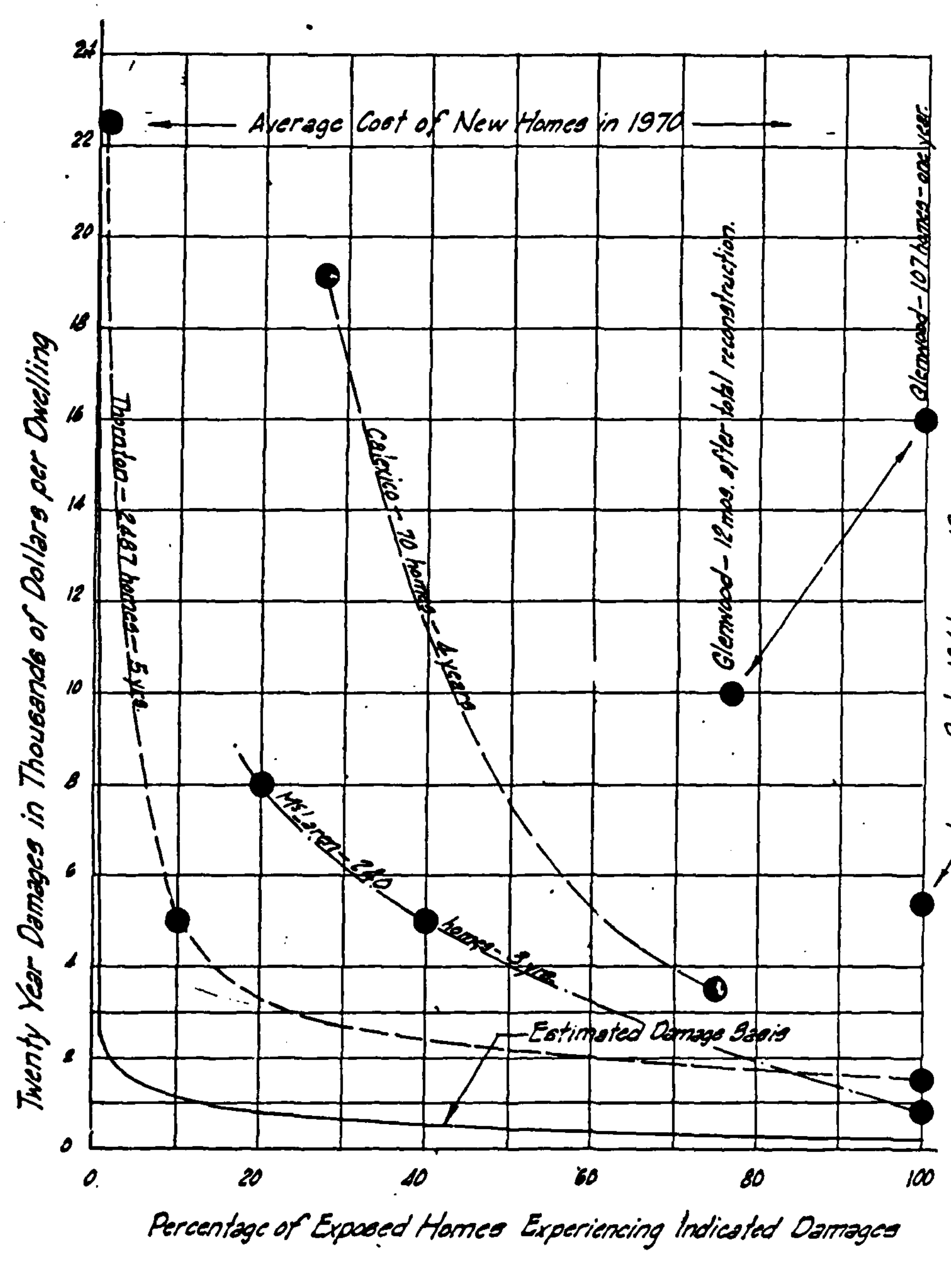

Figure 1. Estimated versus Experienced residential expansive soil damages

Repair of Facilities

13. Mr. William R. Stroman indicated that repair is an unhappy and costly situation. A thorough investigation is required to find the cause, determine extent of damages, design and formulate repair procedures, and deal with the cost. The cause may be continuing, stopped, or reoccur later. The repair must be compatible with the existing structure. Three case histories were reviewed:

a. Fort Polk Airfield Night Lighting Vault. This structure has a simple 20- by 40-ft rectangular plan with concrete masonry unit walls supported on a ribbed mat with perimeter beams 18 inches below the outside grade. The building site is on the side of a hill with the long axis perpendicular to the grade. Approximately 8 ft of fill was required to bring the rear elevation to foundation grade. The fill was soft from poor compaction; the rear of the building had settled about 2 inches. Damage to the foundation was minimal, but a metal clad security door binds and cracks appeared in the walls supported on the loosely compacted fill. Future settlement was minimized by intrusion grouting. Repair cost was $8,700 compared to an original foundation cost of $4,100.

b. Lackland Dental Clinic. Movement of the beam on drilled shaft foundation supporting this rectangular single story brick structure progressed as a wave from one end to the other in the long direction for several years. Damages included cracks in exterior walls and compression and bending failure of the concrete pedestals (plinths) extending from top of the shafts to the underfloor beams. Repairs included installation of short steel stub columns on the top of the shafts to support the superstructure and digging out void spaces beneath the grade beams. These repairs were not made all at one time because of the progression of damage over the structure. Repair cost was $100,000 compared to an original foundation cost of $100,000.

c. Family Housing Project. Aliamanu family housing in Hawaii was constructed in the caldera of an extinct volcano on altered volcanic material of high montmorillonite content. Many units developed moderate to severe damage to the superstructure and foundation from differential movements of up to 4 inches. The 26.5- by 34.5-ft rectangular superstructure of these units is supported by a single perimeter beam 12 inches wide by 18 inches deep with one No. 6 bar top and bottom and No. 3 stirrups on 18 inch centers. Floor slabs are 4 inches thick and they are unreinforced. Repairs include cutting the structure from its foundation and jacking. Another repair procedure is cutting of the floor out of the structure, replacing with a ribbed mat inside the perimeter and tying the whole unit together with post-tensioned cables or conventional steel dowels. Repair costs were $20,000 to $40,000 per unit compared to an original foundation cost of $1500 per unit. The original design of these units was good, but it was replaced with a poor design because of budget and scheduling restraints.

Repair of Mat Foundations

14. Mr. Martin Prager, President of BPR Grouting, indicated that construction and repair must deal with minimizing cost, yet make the structure acceptable. Repairs are often not guaranteed. They should restore the structure as it was and not simply stabilize the soil foundation. Damages often occur from root systems, poor drainage, and construction on sites with little known geotechnical data. Types of repair include underpinning with cast-in-place bored shafts, leveling of slabs by mudjacking, and leveling/raising walls by jacks. Drainage improvements such as grading, placement of downspouts and splash blocks, and drain lines are also made. Repairs can be a substantial portion of the cost of a house. Repairs from settlement damages may be relatively inexpensive at $6,000 to $8,000, while heaving soil repairs can range from $40,000 to $50,000. Improved communication is required to keep up with new technologies.

Review of Handbook for Building Foundation Control During Construction

15. This handbook (see Table 4 for the table of contents) is in preparation and provides recommended procedures and guides for inspectors and people concerned with building construction and implementation of quality control. This handbook was prepared by Dr. K. A. Tenah, Texas A&M University, and Mr. R. L. Crisp, Consultant, for the WES. Final revisions are currently being incorporated into this manual. It will be available after publication from USACE Publications Depot, 2803 52nd Avenue, Hyattsville, MD 20781-1102 (AC 301/436-2063). Professor G. W. Clough recommended that this handbook be offered in an abbreviated version. Mr. Robert Crisp, consultant, indicated that designs and materials are usually adequate, but construction tends to be

Table 4

Table of Contents for Handbook for Building Foundation Control During Construction

Table 4 (Concluded)

inefficient (sloppy) and requires close supervision by inspectors. The inspector is also in the best position to detect any design and construction deficiencies and significant variations of actual soil and site conditions from those assumed for the design. Inspection is therefore most important to be sure that work complies with specifications and completes the intent of the structure.

16. Mr. Crisp stated that Corps of Engineer construction is often driven by scheduling and time constraints. Inspectors are therefore sometimes given jobs in which design details of the foundation and superstructure have not been completed due to these constraints and uncertainties in actual field conditions. The inspector must arrange with the designers and contractors for completion of plans that will fulfil the intent of the structure. Inspectors are often among the least paid people in the Corps; however, they are given responsibilities comparable with highly trained professional engineers.

SESSION III: ACTIVE RESEARCH

Contributions From Academic Community

17. Professor Wray initially reviewed sources of expansive soil studies at several universities such as University of Texas at Austin, University of Texas at Arlington, University of Missouri at Rolla, Colorado State University, Case Western, Syracuse, Texas A&M, University of California at Berkeley, University of Illinois at Urbana, Texas Tech, and others. Most publications may be found in the 5 international conferences conducted on expansive soils. Short courses have been provided by the PTI, Corps of Engineers, American Concrete Institute, and Colorado State University. Refer to the publication of the United States Universities Council on Geotechnical Engineering Research (USUCGER) available from Professor G. Wayne Clough, Virginia Tech, for further information.

18. Professor Wray reviewed a field study at an Amarillo, TX site, sponsored by the National Science Foundation (NSF) which consists of a 36- by 52-ft plastic membrane covered by a thin sand layer. A grade beam is located at one end. Edge heave had been recorded at this site with some center heave beginning to occur. Edge heave has not fluctuated significantly with time within the end containing the grade beam. Edge heave fluctuations have been significant at the end without the grade beam. Heave had been observed primarily down to 3 or 4 ft with some as deep as 9 ft. Suction changes indicate an active zone for heave down to about 4 or 5 ft. Suctions tend to be relatively high at this site, perhaps associated with dry periods. This test section demonstrates the importance of field tests to determine appropriate soil input parameters for design.

19. Mr. R. Gordon McKeen, Senior Research Engineer, New Mexico Engineering Research Institute, described research associated with earlier Federal Aviation Administration (FAA) work to develop a design procedure for runways. Most problems with runways appear to be associated with utility trenches and possibly swelling backfill. One important part of this work is demonstration of a new frequency spectrum model originated by Professor Robert L. Lytton, Texas A&M University, for characterizing heave patterns by a sine wave. The wave pattern of heave appears to be rooted in the natural crack structure. Design guidelines have been developed for airport runways based on developing an acceptability criterion between the amplitude and wavelength of the soil

distortion pattern. The structural requirements of the runway are evaluated through the beam on a Winkler foundation concept. Mr. McKeen is working on an initial project under a Broad Agency Announcement contract with the WES to adapt the FAA work to building foundations. This work should be completed near the end of this year.

Contributions From Corps

20. Mr. Hartman, SWD, has developed a computer program RIBMAT2 for non-linear soil structure interaction analysis based on the beam on Winkler foundation concept. This program with other two and three dimension finite element programs help provide a good idea how a mat will perform. Some analysis was directed to finding the effective width of the slab appropriate for design of a T-section. The T-section width is usually taken as the sum of the rib width plus slab thickness on each side of the rib. Analysis has shown that column loads are distributed because of stiffness in the beams and that the soil swell profile has significant impact on mat behavior. Mr. Hartman indicated that more work is needed to better characterize soil behavior for input into design programs and for predicting actual mat behavior. The frequency spectrum model looks good because it can consider other foundation systems as well as mats and information on the probability of degrees of damage can be provided. Mr. Hartman indicated that economical and reasonable structural designs for edge movements exceeding 1 inch are difficult, perhaps not even practical, to achieve.

21. Mr. Al Branch indicated that about 900 mats have been constructed by the FWD, mostly in smaller sizes, with reasonable performance. Complaints amount to less than 1 percent of the ribbed mats. Some guidelines for soil input parameters have been developed for use with the SWD Design procedure and included in the report, Appendix A. Most problems with damages have been associated with unusual point sources of wetting. It is also important to design the flexibility of the superstructure to be compatible with the foundation movements. Mr. Stroman suggested that these experiences and the associated economic costs of these foundations should be compared with those at the Aliamanu Family Housing project.

22. Mr. Chris Merrill of the Information Technology Laboratory reviewed the Computer Aided Structural Engineering (CASE) project. The CASE program is just beginning to initiate studies with mat foundations. WES publications available to date include:

a. Technical Report K-85-1, "Application of Pasternak Model to Some Soil-Structure Interaction Problems", Volumes I and II, by A. D. Kerr.

b. Technical Report ATC-86-1, "Foundation Interaction Problems Involving an Elastic Half-Plane", by H. B. Wilson and L. H. Turcotte.

c. Technical Report ATC-86-2, "The Application of Boundary-Element Techniques for Some Soil-Structure Interaction Problems", by C. V. G. Vallabhan and J. Sivakumar.

d. Technical Report ATC-86-3, "Soil-Beam Interaction Analysis With a Two-Parameter Layer Soil Model: Homogeneous Medium", by T. Nogai and L. C. Lam.

23. Dr. Lawrence Johnson described a field investigation of a mat at Red River Army Depot, TX. One purpose of this study is to help develop guidelines for design input parameters. The foundation is a large and heavily reinforced mat 675 ft by 300 ft with stiffening ribs 36 inches deep by 18 inches wide. Rib spacing is 12.5 ft near the perimeter and 25 ft interior. Rib reinforcement steel consists of two No. 11 bars top and bottom. The mat was designed for maximum edge lift of about 1.5 inches.

24. Laboratory and field investigations were conducted to characterize soil behavior. The mat was constructed on a nonexpansive compacted fill of 4 to 6 ft deep. Expansive soil of the Upper Midway formation is found below this depth. Piezometers indicated a hydrostatic perched water table 5 to 7 ft below ground surface continuous to below 40 ft in depth. The potential swell at this site is, therefore, essentially negligible as confirmed by the settlement profile of the level surveys. Readings from earth pressure cell and strain gage instruments placed in one stiffening beam are consistent with results of the level surveys; the behavior of the ribbed mat is consistent with a plate on a semi-infinite elastic foundation. Settlements approach 0.2 inch except beneath an expansion joint where settlement approaches 1.5 inches. This large settlement coincides with the location of an old drainage ditch prior to construction. This study shows considerable potential for future savings through understanding of site characteristics and soil deformation patterns prior to design and construction. Thorough site studies permit efficient design of the foundation to fit the expected deformation pattern. Geotechnics is often the least understood area of a project. Mr. Stroman suggested that this structure should be monitored for at least 5 more years

because this time may be required for the structure to respond to movement of the deeper expansive soil beneath the thick non-expansive fill supporting the mat.

Problems in Need of Research

25. Professor Wray reviewed the results of a steering committee meeting conducted 29-30 June, which indicated areas of required research, Appendix B. The most important finding is that research should be directed toward proper soil characterization and understanding of the performance of soil supporting the foundation.

SESSION IV: PROPOSED RESEARCH PLAN

Presentation of Basic Plan

26. Professor Wray presented a proposed plan of study shown in Appendix C which he prepared as part of an Interagency Personnel Agreement with WES. Field investigations are required to determine proper soil characterization and selection of suitable input parameters for design and construction of mat foundations. A need to reeducate engineers is necessary to accomplish proper analysis and design. It was emphasized by the participants that in situ tests such as the mini-cone or piezocone to complement soil sampling should be considered for field studies. Characterization of preconstruction conditions should also be included in the study. Mr. McKeen indicated that climate is a dominant factor in slab behavior as shown by the FAA and NSF studies and a long term project should be planned in order to experience representative cycles of climate.

Historical Perspectives and Future Directions

27. Professor Robert L. Lytton, Texas A&M University, presented requirements for a comprehensive plan: sound theory, field observation, laboratory measurement, empirical relationships, and risk formulation. One major problem with respect to soil is to characterize movement in the field. The worst cases of differential movement should be evaluated to characterize soil behavior and evaluation of maximum bending moments and shears in the mat. Dr. Lytton described experiences with gilgai which show heave patterns characterized by wave-lengths of vertical movement with peaks at about 17 ft. One-half the mean amplitude can be related with frequeny of peaks in the bumps. The pavement or foundation filters out the waves. A roughness spectrum of a plot of β

$$\beta = \sqrt[4]{\frac{kb}{4EI}}$$

where

β = beam stiffness, ft^{-1}

k = coefficient of subgrade modulus, $kips/ft^3$

b = width of beam, ft

E = elastic modulus of pavement, $kips/ft^2$

I = moment of inertia of pavement, ft^4

versus the ratio of wavelength of pavement/wavelength of soil has been developed for pavements. A similar pattern probably exists for buildings, although few, if any, spectrum measurements have been made for these facilities. The amplitude versus wavelength or roughness spectrum should be determined for buildings and correlated with the condition of the building. The mat foundation will be required to span between peaks of the wavelengths.

28. Additional information provided by Dr. Lytton is as follows:

a. Previous Slab Design. BRAB uses a cracked section which results in costly overdesign for ground slabs. The cracked section does not consider the strength of the concrete slab to resist bending. The uncracked or gross section should be adequate for slabs placed in the ground so that the concrete resistance to bending will be included. The design should consider the envelope (maximum and minimum distributions) of soil induced, uniform and column loads.

b. Site and Soil Characterization. This may be accomplished by recording cracks and seams in soil by non-destructive testing, use of Draeger tubes, in situ cone testing and measurements of electrical conductivity and pH. Measurements of the roughness spectrum inside and outside of buildings should be most useful. Boundary (envelope) values of moisture changes are necessary. Slabs may be characterized by a two-dimensional (anisotropic) roughness spectrum and analysis of stiffened slabs and underreinforced slabs with consideration of soft spots. An accept-ability criterion can be developed using the roughness spectrum and correlation with damage records. Soil-structure interaction analysis can be accomplished with personal computers for beam or slab on curved mounds. Moment, shear, and deflection envelopes should be determined. A rationally established level of risk must be determined for mat foundations. These studies can be accomplished with limited resources.

Working Groups

29. Working groups were organized to answer specific questions developed by Dr. Wray given in Appendix D to help develop the proposed research plan further. Answers to these questions, Appendix D, show that the first priority of work is field investigations. Other important work includes pulling together all available data on design and construction into a "clearing house" for distribution. The frequency spectrum approach to design of mats should be pursued and include anisotropic roughness measurements and modification of the

spectrum model for pavements to consider mats. A systematic performance record system for structures must also be developed prior to measurement of the roughness spectrum inside and outside of buildings.

30. Results from studies conducted under the research plan should be distributed through publications at WES, technical journals, and international and specialty conferences. The developed methodology should become widely accepted in time with proven records of construction economy and successful performance. Assured widespread use requires regulations and codes to impose design and construction methods or any other procedure; however, implementation of codes may not be practical.

Instrumentation

31. Professor G. Wayne Clough, Virginia Tech, and Dr. Klaus Hilmer, Chief of the Soil Research Institute in Nurnburg, West Germany, reviewed experiences and case histories with field instruments. Most instruments were earth pressure cells, extensometers, and inclinometers. Computer simulations should be used to predict vertical and horizontal movements and then compared with field measurements. Earth pressure cells should not be inside or in contact with concrete, but should be isolated with a soft material such as mastic to avoid bending moments in the cells. Additional comments are as follows:

a. Dr. Hilmer provided several handouts on case histories to the participants. He has the laboratory capability of prototype model testing to investigate horizontal and lateral earth pressures. Records indicate zero failure rate for use of field instruments up to 10 years. He uses Gloetzl cells to measure earth pressures; these thin cells are placed on 3 inches of sand.

b. Dr. Clough provided the following rules of instrumentation: know the geotechnical aspects of the problem, keep the dirt (geotechnical engineer) guy in the team, assure redundancy of instruments, and concentrate instruments at single locations. Readings should be made relative to causative events and not convenient schedules. Accurate and detailed written and pictorial records should be kept. Instruments should be evaluated for reliability and ruggedness. Long established reliable instruments such as levels for elevation surveys and inclinometers should be used as well as new instruments.

Field Demonstration Concepts

32. Mr. Branch reviewed field study concepts in preparation of full scale implementation of instrumented mats for developing guidelines for

evaluation of soil behavior and soil input parameters for design. Field test sections are proposed to be constructed, perhaps some in Fort Sam Houston, San Antonio, TX. The size of the mats should be about 40 ft by 60 ft, which is characteristic of the most common mat dimensions, and include conventional reinforced and post-tensioned mats. Soil stabilization with chemicals and moisture barriers should be considered and some sections should be subject to ponding. The sites should be thoroughly characterized through use of cone penetration, pressuremeter, and laboratory soil tests on undisturbed specimens. Test sections should be loca-ted in different climates and soil conditions to evaluate environmental effects.

33. Mr. Stroman indicated that serious consideration should be given to the effects of ground modification. The data that Dr. Poor and others at the University of Texas at Arlington produced several years ago should be reviewed and analyzed since its full importance may not have been realized. Following this, the field research program can be adjusted to evaluate performance with no soil modification, soil modified by lime injection, soil modified by plain water injection, and soil modified by the potassium product. Results of these studies may be especially important if, as Mr. Hartman indicated in paragraph 20, the structural community is stymied by differential edge movements greater than 1 to 2 inches.

34. Possible sources of funding for such a program include regional district/division offices for sponsorship of a test section in their own area. Maintenance funds may be a potential source of funds, particularly for family housing interests who have been especially hurt from damages of structures on expansive soil. The research plan must be fully documented with potential savings. Mr. Robert Yunker, FM&S Branch, Pacific Ocean Division, indicated that a public relations type information, education, and solicitation film could be prepared to show customers for the purpose of obtaining financial support. The Corps will be obligated to provide timely and complete state-of-the-art pro-grams and design procedures usable by any design and educational institution.

Summary

35. Professor Walter C. Sherman, Tulane University, indicated that this workshop was especially valuable in bringing people together to discuss important aspects of design and construction on expansive soils. The most common

and effective mat design procedures include those of the SWD and the PTI. The session on construction and performance indicated that repair of structures damaged by expansive soil movements is costly and can easily exceed the original foundation cost. Proper construction quality control is important for good performance and requires adequate inspection.

36. A new and simple approach to mat design using frequency spectrum technology was suggested at this workshop. The frequency spectrum technology was originally developed for design of pavements and not yet applicable to mats. Analysis of the roughness spectrum inside and outside of structures should be completed and coupled with performance records; this information can lead to acceptable and unacceptable criteria for mat performance and an efficient design methodology. More work was especially indicated in site characterization through a field study program. The field program would be useful to evaluate the depth of active zones for heave, edge moisture variation distances, maximum and minimum wetting profiles, and effectiveness of soil modification treatments. In summary, the workshop provided a valuable medium to develop the status, deficiencies, and goals for mat foundation research.

APPENDIX A:

DEVELOPMENT OF DESIGN FORMULAS FOR RIBBED MAT FOUNDATIONS IN EXPANSIVE SOILS

by

Joseph P. Hartman

SWD REPORT

DEVELOPMENT OF DESIGN FORMULAS FOR RIBBED MAT FOUNDATIONS IN EXPANSIVE SOILS

by

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US ARMY CORPS OF ENGINEERS
SOUTHWESTERN DIVISION
DALLAS, TEXAS

DECEMBER 1986

Table of Contents

1. INTRODUCTION.

This report contains background information which led to the development of design formulas presented in Exhibit 1. These formulas apply only to structural design of ribbed mat foundations on expansive soils. Previous design formulas were judged to be inadequate for general application within Southwestern Division. The new formulas were developed to provide an adequate design method, other than performing a nonlinear soil-structure interaction analysis. Such computer analyses were used, however, to provide the basis for development of the new formulas. These analyses were performed by Tulsa District, Structural Section, under the direction of the group listed in Appendix A.

2. COMPUTER ANALYSIS.

2.1. Computer Program. The program used to analyze a ribbed mat foundation was CBEAMC (reference 5.2). This program was used to analyze a model consisting of a beam supported by nonlinear springs.

2.2. Computer Model.

2.2.1. Beam. The beam used in the computer model represented the smeared bending stiffness of a 1 foot strip of a typical ribbed mat. The beam extended from the perimeter, 30 feet towards the interior of the mat. Symmetrical boundary conditions were applied at the interior end. Such end conditions are appropriate since results indicate that perimeter soil behavior has little effect at that distance. Parameters used to describe beam stiffness included the effective rib moment of inertia (I) and the rib spacing (s). The smeared stiffness (I') was taken as I'=I/s. The effective moment of inertia may represent the bending stiffness of a tee beam formed by a rib plus an effective width of slab acting as a top flange.

2.2.2. Soil. Soil support for the mat was represented by non-linear Winkler springs. Stiffness of the springs for downward displacement was dependent on the assumed subgrade modulus (k); upward displacement would result in loss of contact between mat and soil. The basic spring behavior is shown in Figure 1. Near the exterior end of the beam soils would be subject to moisture-induced volume changes. Soil shrinkage would result in loss of support near the perimeter, this condition is referred to as center lift. Soil swell would result in lifting of the perimeter of the mat, this condition is referred to as edge lift. The extent of soil shrinkage or swell is defined by the edge moisture variation distance (Lm), and the magnitude of shrinkage or swell is defined by "soil heave" (Ym). These parameters are more fully described in Exhibit 1.

For the center lift condition spring definitions included an offset (DO), which represents the potential soil shrinkage due to moisture changes if no significant loads are applied to the soil. This is shown in Figure 2. For the edge lift condition the offset (DO) represents the potential expansion of the soil if no loads are applied. However, the expansive potential was limited to an assumed maximum interface pressure (Psw) between the mat and the soil. This perimeter spring behavior for edge lift is shown in Figure 3.

2.2.3. Loading. Loads applied to the beam consisted of a uniform distributed load (p), a concentrated load at the perimeter (Pp), and a concentrated interior load (Pi). The interior load was located at a varying distance (Li) from the perimeter.

2.2.4. Parameter Values. A typical range of values was identified for each of the parameters identified above, and a baseline (most common) value was selected. The selected parameter values are given in Table 1.

2.3. Analyses. A computer analysis was performed using the baseline value for each parameter. Additional analyses were then performed by changing the value of a single parameter while retaining all other baseline values. This procedure was followed for both center lift and edge lift conditions.

3. ANALYSIS RESULTS.

3.1. Numerical Results. Numerical results of each analysis are presented graphically in Appendix B. Important design results include maximum deflections, moments and shears. It can be seen that these are affected to differing degrees by variation of each parameter.

3.2. Physical Analogies. A review of the results will indicate that for center lift the end of the beam behaves much as a pure cantilever. For edge lift the outer portion of the beam behaves similar to a simply supported beam where one support has been raised slightly. Development of design formulas was based on this cantilever and simple support behavior.

4. DESIGN FORMULAS.

4.1. Objective. The objective was to develop design formulas which were simple, accurate, rational and flexible. Flexible indicates that the formulas should be applicable to a wide range of problems. Rational indicates that the formulas should make sense physically to a designer, rather than be a mysterious "black box."

4.2. Center Lift. Formulas for center lift design are included in Exhibit 1, Part II, paragraph 3.1. The first step is to determine the length of an equivalent cantilever beam. Once this is done the designer uses conventional formulas to determine moments and shears in the cantilever. For deflections, additional adjustments must be made to account for the fact that the support for the cantilever is not truly fixed. The cantilever model makes physical sense to a designer, only determination of the proper length is a black box formula.

4.3. Edge Lift. Formulas for edge lift design are included in Exhibit 1, Part II, paragraph 3.2. The first step is to determine the length of an equivalent simple beam, based on an assumed perimeter deflection. Calculated deflection is used to determine a new equivalent length and this process continues until assumed deflection converges with calculated deflection. The iterative process increases the complexity of the method, but is unavoidable if accuracy and flexibility of the formulas are to be achieved. Once the equivalent simple beam length is determined the designer calculates moments and shears by conventional formulas. The simple beam model again makes physical sense to the designer and calculation of edge deflection is based on a rational approach, only determination of the proper length is a black box formula.

4.4. Verification of Formulas. To demonstrate the accuracy of the formulas, Tables 2 and 3 show comparisons of computer results with formula results, for maximum moments and displacements. The comparisons demonstrate sufficient accuracy of the formulas. However, use of parameter values outside the range of those used in the computer analyses, or combinations of non-baseline values for several parameters, will inevitably result in larger differences when comparing formula results to computer solutions. It should be noted that the formulas are intended only to match the computer results, therefore, adequacy of the formulas is limited by adequacy of the computer model, especially the method used to represent soil behavior. Idealization of soil and structural behavior is fairly crude and should be improved through further, more detailed investigations.

5. REFERENCES.

5.1. Letter, SWDED-TS/G, 23 Dec 1986, "Design Criteria for Ribbed Mat Foundations". (Exhibit 1 is an enclosure to this letter)

5.2. Instruction Report K-82-6, "User's Guide: Computer Program for Analysis of Beam-column Structures with Nonlinear Supports (CBEAMC)", US Army Waterways Experiment Station, June 1982.

Table 1 - Parameter Values Used in Computer Analyses

Parameter	Center Lift	Edge Lift
Lm (ft)	2 <u>5</u> 8	2 <u>5</u> 8
Ym (in)	.5 <u>1</u> 2 3	.5 <u>1</u> 2 3
k (pci)	50 <u>100</u> 200	50 <u>100</u> 200
Psw (psf)	NA	<u>2</u> 4 8
I (1000 in^4)	15 <u>30</u> 60 120	15 <u>30</u> 60 120
s (ft)	12 16 <u>20</u> 24	12 16 <u>20</u> 24
Pp (klf)	1 <u>3</u> 5	0 <u>1</u> 3
Pi (klf)	<u>0</u> 3 5	0 <u>3</u> 5
Li (ft)	<u>16</u>	6 12 <u>16</u> 20
p (psf)	<u>100</u>	<u>100</u> 250

Note: Baseline values are underlined

Table 2 - Comparison of Center Lift Results

Parameter	Formulas M(ft-k)	Formulas D(in)	Computer Mc(ft-k)	Computer Dc(in)	Comparison M/Mc	Comparison D/Dc
Baseline	13.6	.324	13.2	.32	1.03	1.01
k=50	13.6	.413	13.2	.41	1.03	1.01
k=200	13.6	.261	13.2	.26	1.03	1.00
Ym=0.5	12.5	.284	12.5	.27	1.00	1.05
Ym=2.0	14.9	.374	15.6	.36	0.96	1.04
Ym=3.0	15.7	.408	16.0	.39	0.98	1.05
Lm=2	9.6	.205	9.2	.19	1.04	1.08
Lm=8	17.7	.507	17.1	.54	1.04	0.94
I/s=.75	12.1	.435	12.5	.43	0.97	1.01
I/s=3	15.3	.251	15.9	.23	0.96	1.09
I/s=6	17.3	.203	17.4	.20	0.99	1.02
Pp=1	6.0	.188	6.2	.15	0.97	1.25
Pp=5	20.7	.473	20.8	.47	1.00	1.01

Table 3 - Comparison of Edge Lift Results

Parameter	Formulas M(ft-k)	Formulas D(in)	Computer Mc(ft-k)	Computer Dc(in)	Comparison M/Mc	Comparison D/Dc
Baseline	12.8	.51	11.8	.55	1.08	0.93
Ym=0.5	9.4	.27	7.3	.26	1.29	1.04
Ym=2.0	16.9	.94	18.2	1.00	0.93	0.94
Ym=3.0	19.3	1.35	22.5	1.38	0.86	0.98
Lm=2	6.6	.17	5.7	.17	1.16	1.00
Lm=8	14.7	.66	13.7	.66	1.07	1.00
I/s=.75	7.8	.57	7.1	.60	1.10	0.95
I/s=3	18.6	.45	17.5	.46	1.06	0.98
I/s=6	23.9	.41	24.5	.39	0.98	1.05
Pp=0	14.7	.66	13.7	.66	1.07	1.00
Pp=3	9.1	.27	8.2	.26	1.11	1.04
Pi=0	7.6	.57	7.2	.57	1.06	1.00
Pi=5	14.6	.49	13.7	.53	1.07	0.92
Li=6	12.3	.40	12.2	.34	1.01	1.18
Li=12	15.4	.47	14.7	.48	1.05	0.98
Li=20	9.4	.54	8.3	.54	1.13	1.00
p=250	13.6	.36	10.4	.42	1.31	0.86
Psw=4	15.2	.72	13.3	.65	1.14	1.11
Psw=8	16.4	.85	13.5	.68	1.21	1.25

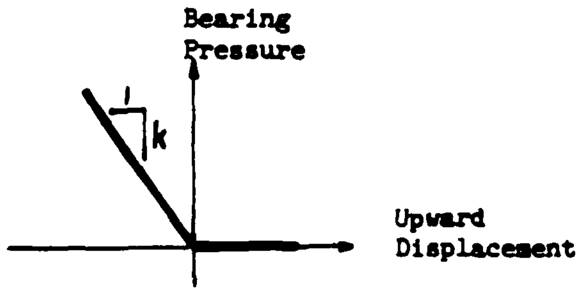


Figure 1 - Basic Soil Spring

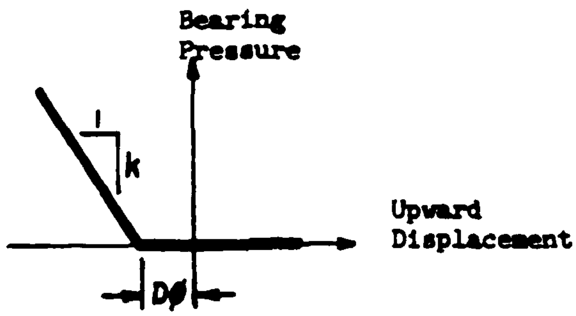


Figure 2 - Spring for Shrinking Soil

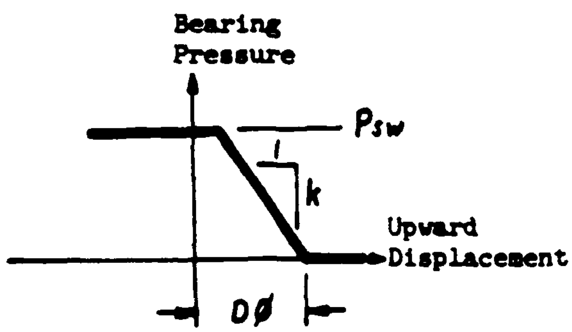


Figure 3 - Spring for Swelling Soil

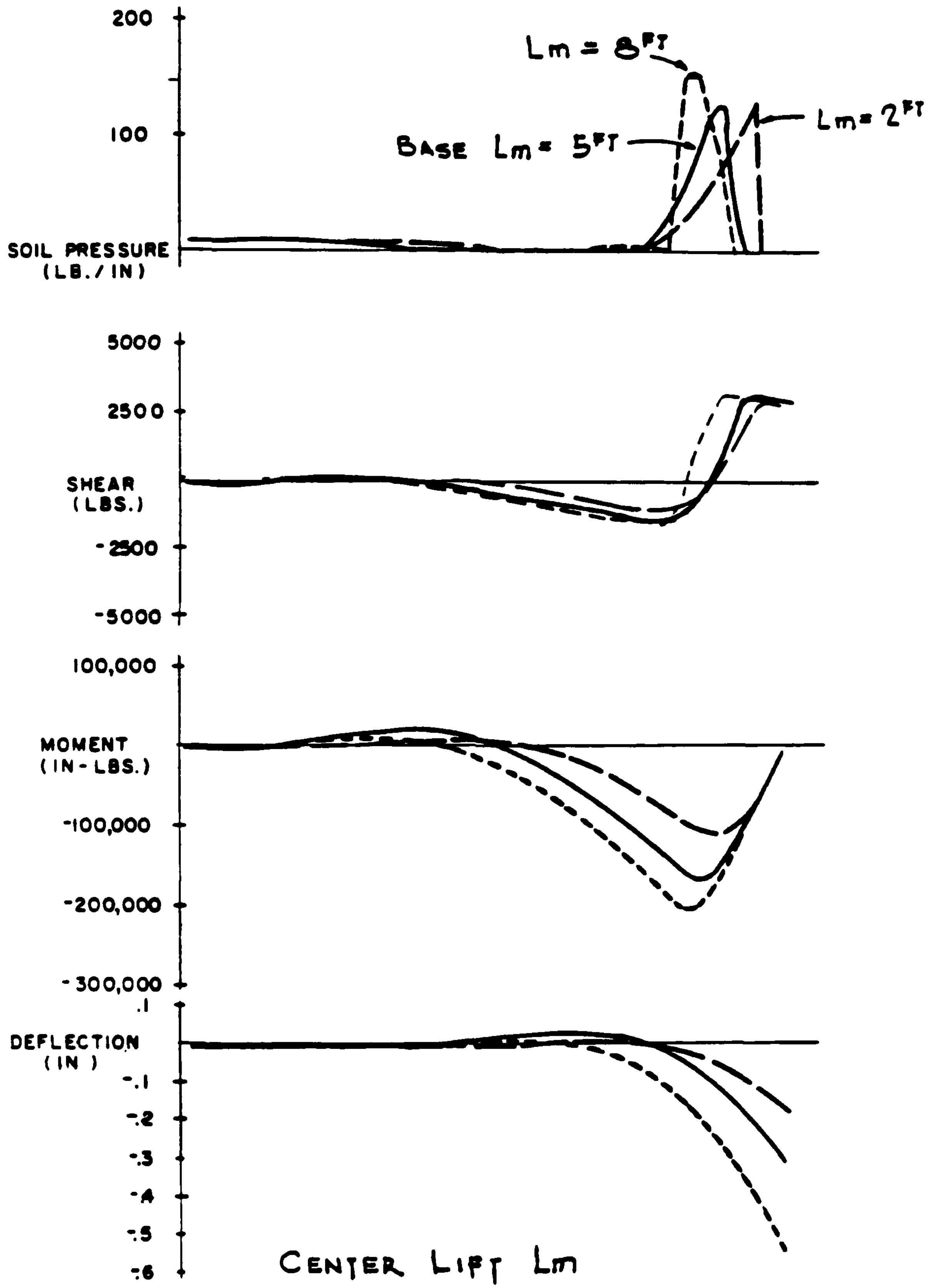
Appendix A

Advisory Group Members

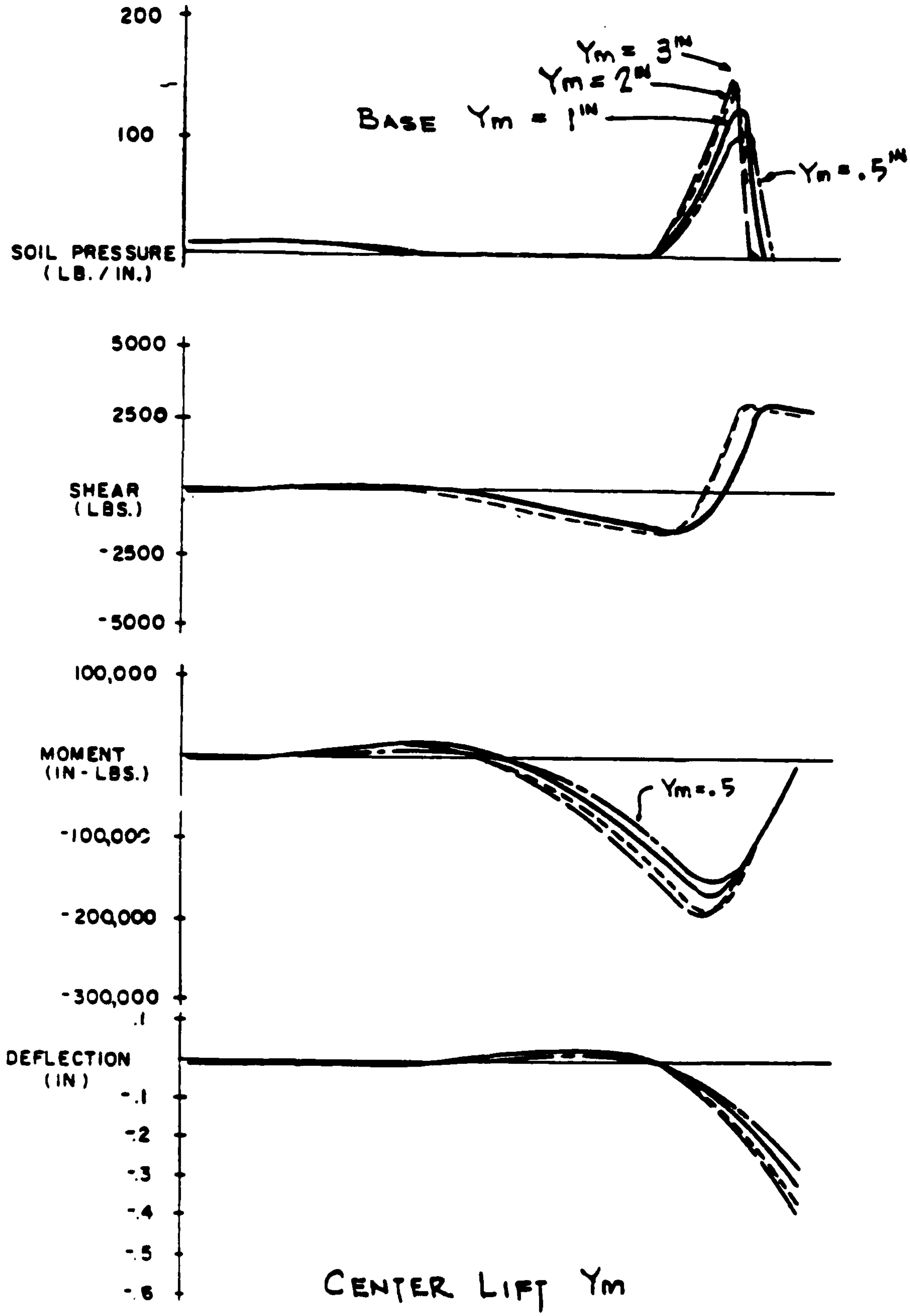
Joseph Hartman	SWDED-TS
Jack Fletcher	SWDED-G
Garland Young	SWFED-DT
Al Branch	SWFED-FD
George Henson	SWTED-DT
Sandy Stephens	SWTED-DT
Harrison Sutcliffe	SWTED-DT
George Hall	SWTED-GP
Cliff Warren	SWTED-GP

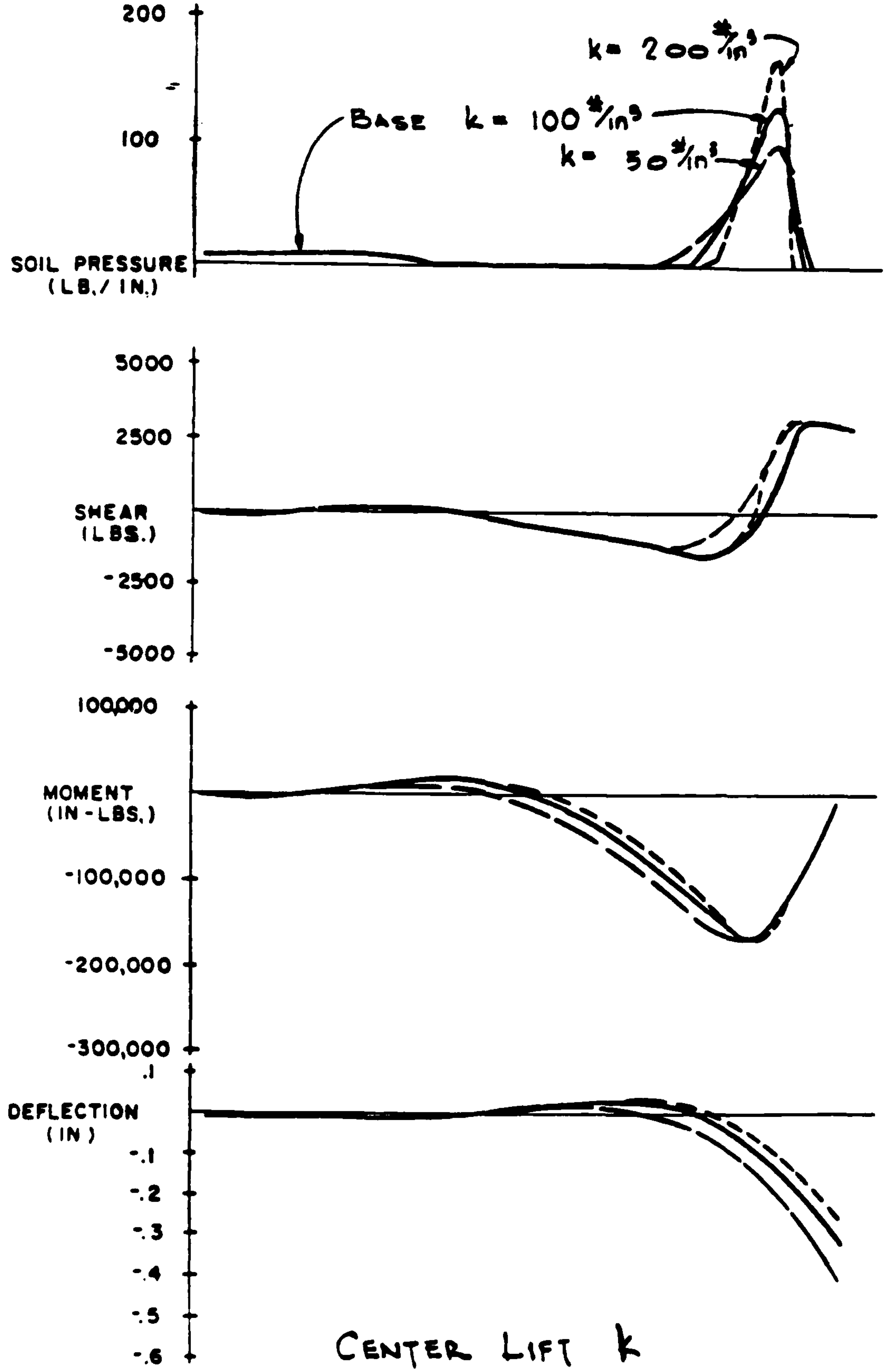
Appendix B

Numerical Results of Analyses

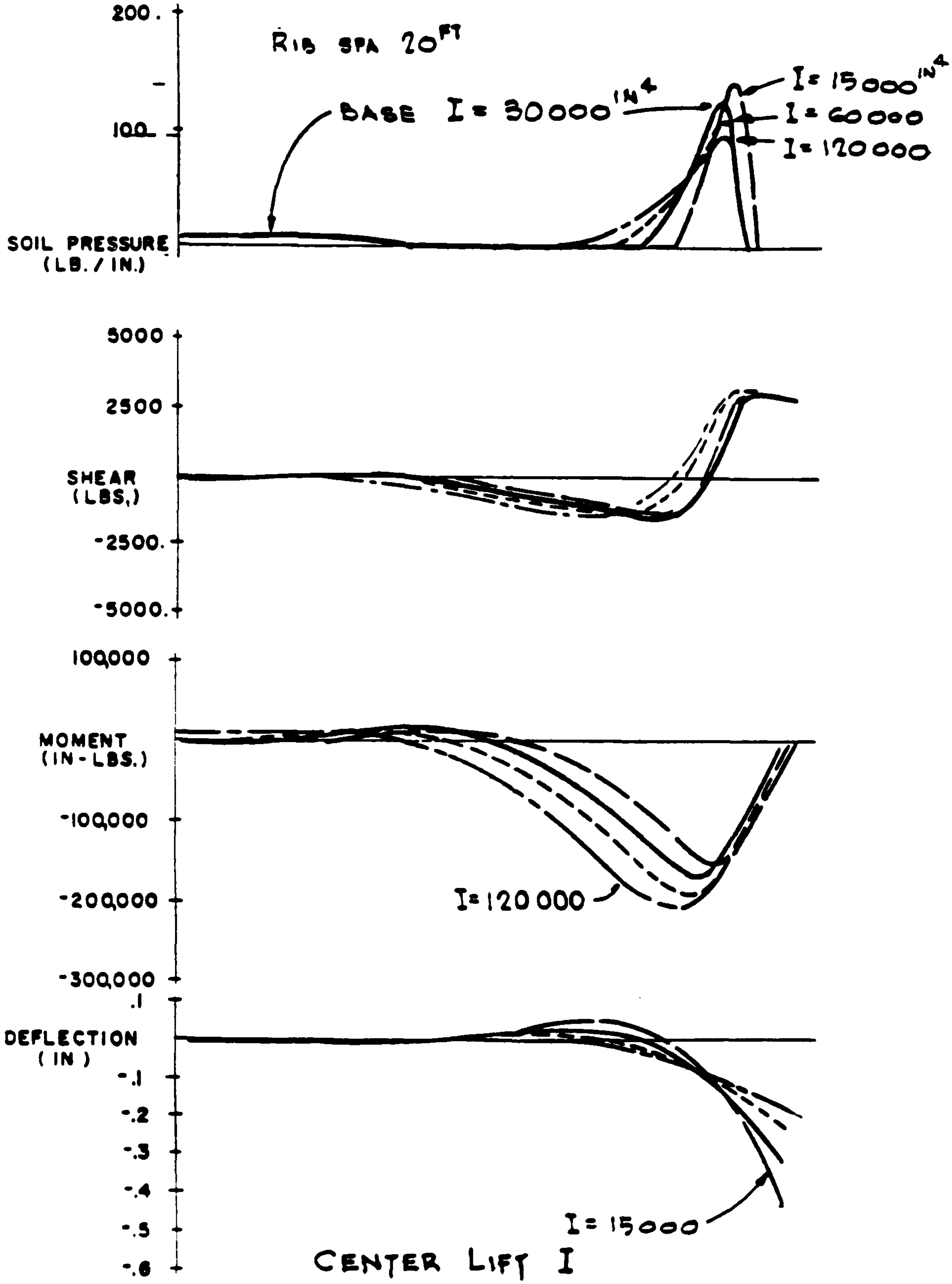


200
100
Lm = 8FT
BASE Lm = 5FT
Lm = 2FT
SOIL PRESSURE
(LB./IN)
5000
2500
SHEAR
(LBS.)
-2500
-5000
100,000
MOMENT
(IN-LBS.)
-100,000
-200,000
-300,000
.1
DEFLECTION
(IN)
-.1
-.2
-.3
-.4
-.5
-.6
CENTER LIFT Lm

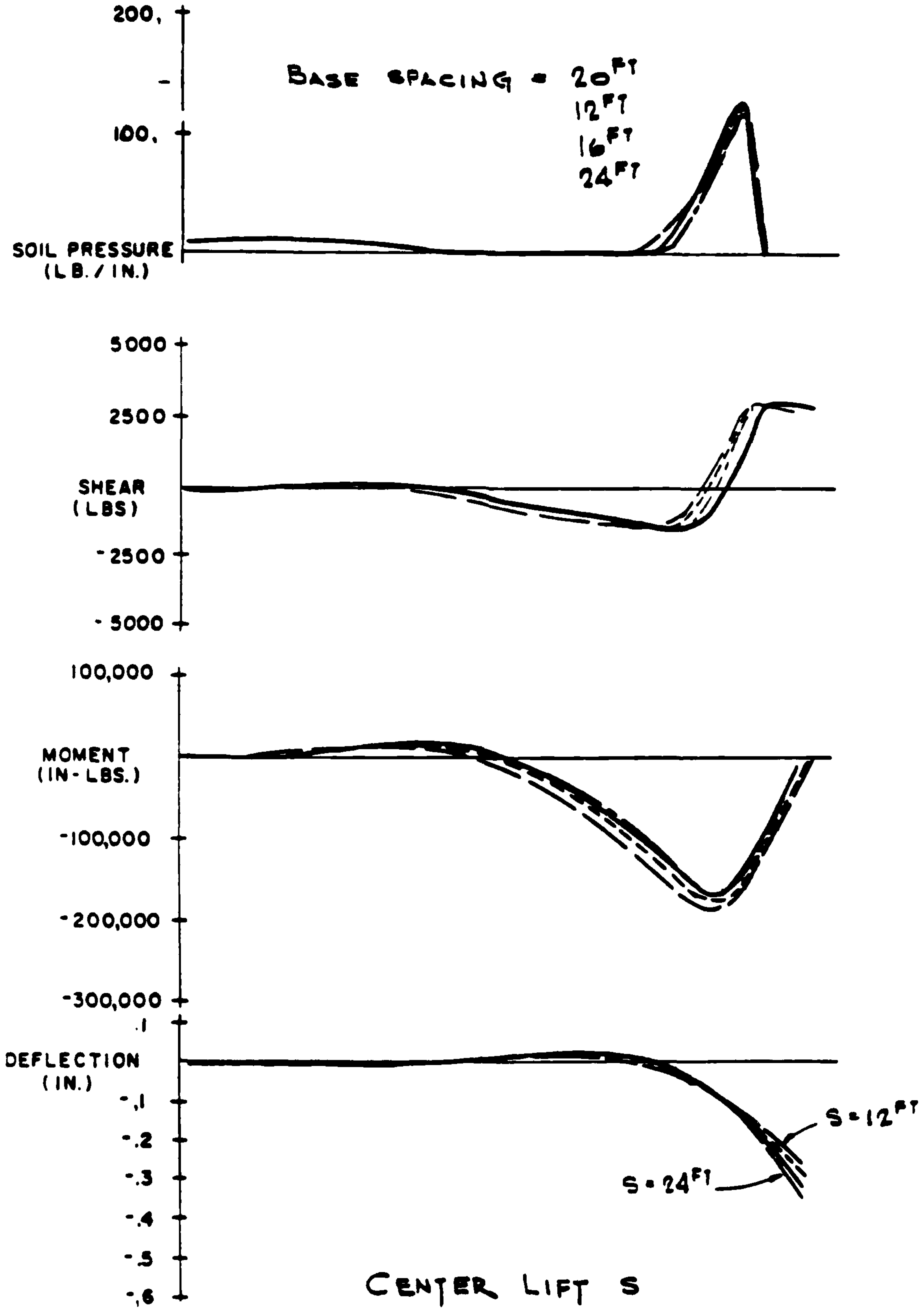

200
100
Ym = 3 IN
Ym = 2 IN
BASE Ym = 1 IN
Ym = .5 IN
SOIL PRESSURE
(LB./IN.)
5000
2500
SHEAR
(LBS.)
-2500
-5000
100,000
MOMENT
(IN-LBS.)
Ym = .5
-100,000
-200,000
-300,000
.1
DEFLECTION
(IN)
-.1
-.2
-.3
-.4
-.5
-.6
CENTER LIFT Ym



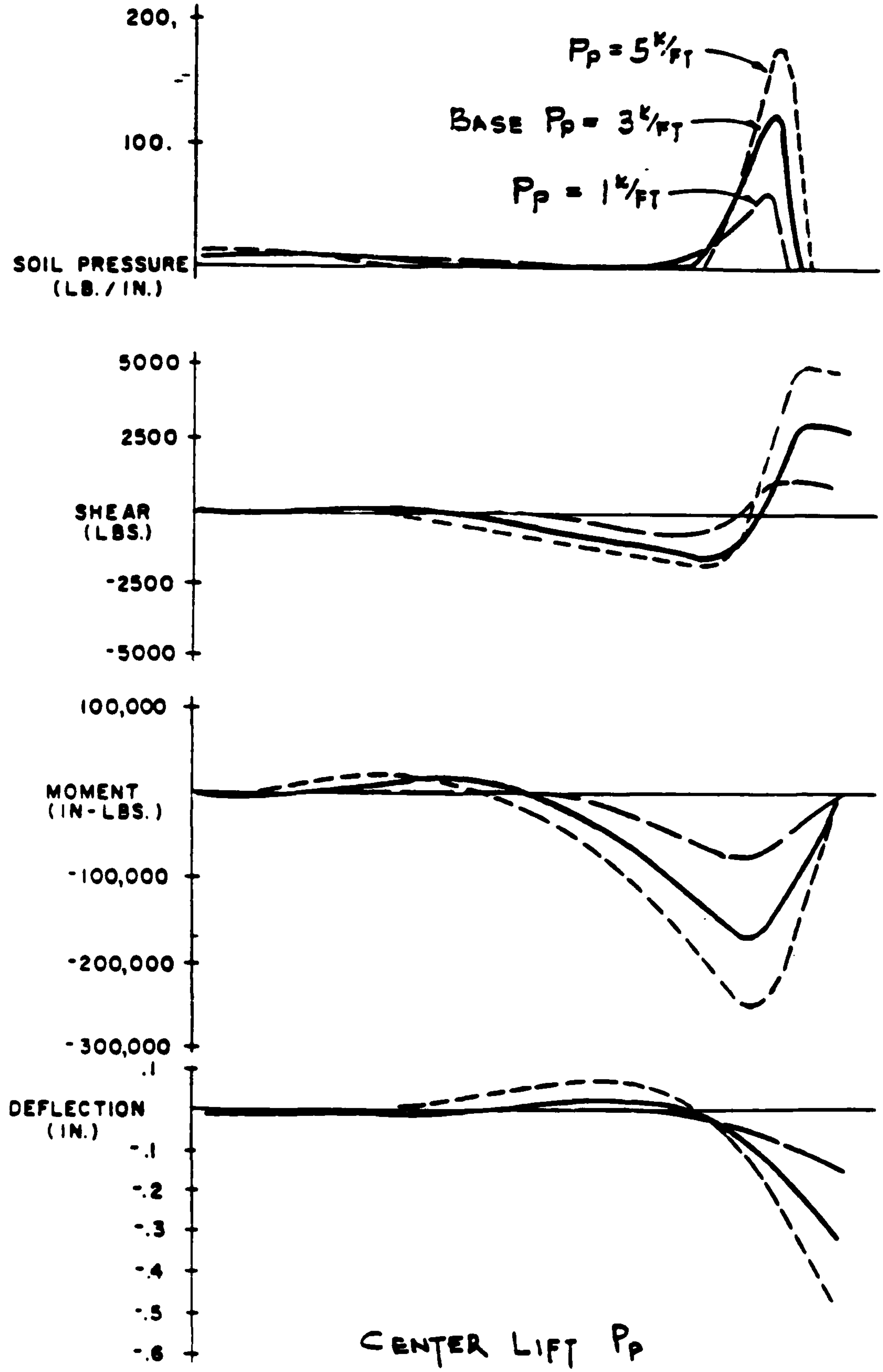
CENTER LIFT k



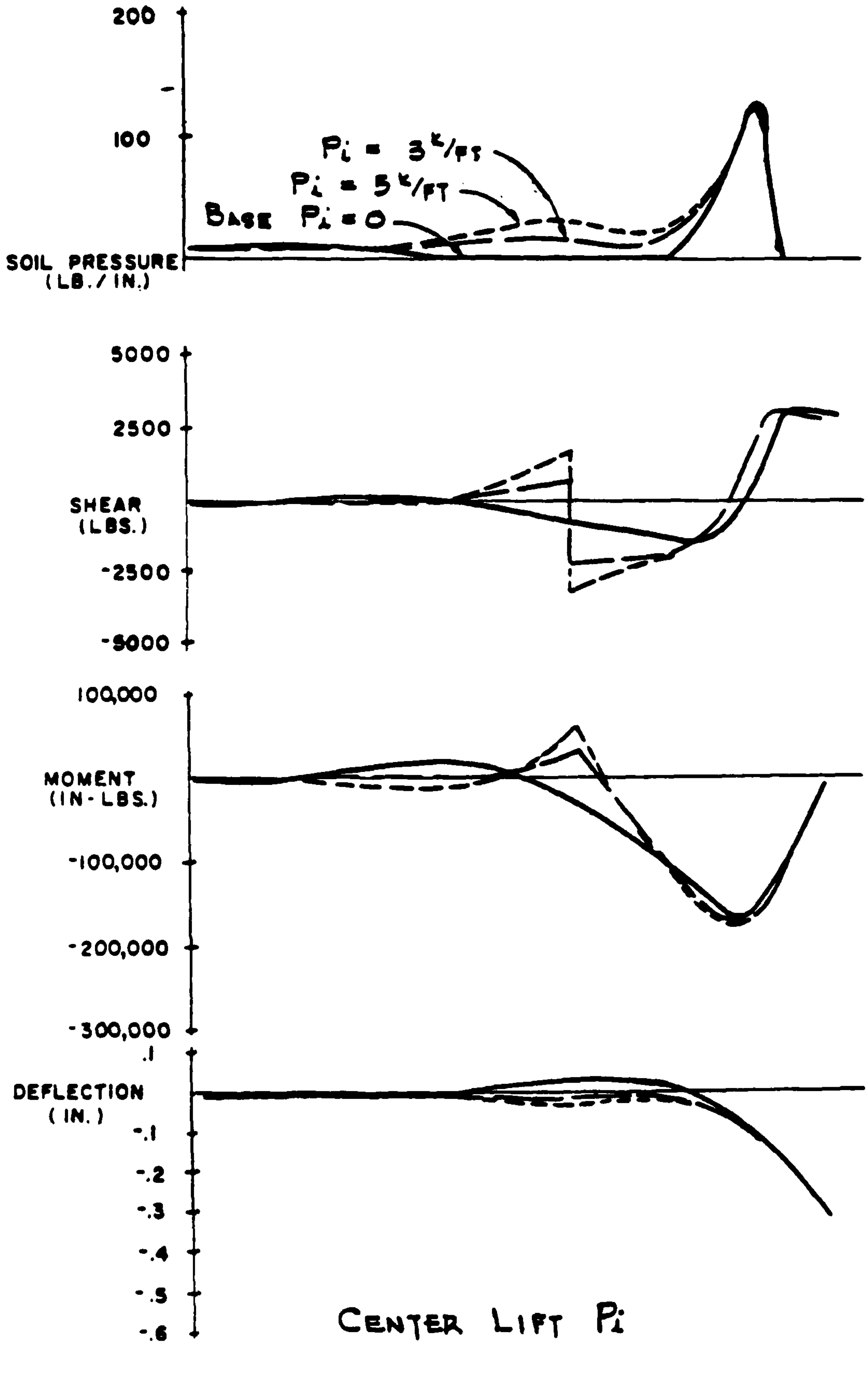
200.
RIB SPA 20FT
I= 15000IN4
BASE I= 30000IN4
I= 60000
100
I= 120000
SOIL PRESSURE
(LB./IN.)
5000
2500
SHEAR
(LBS.)
-2500.
-5000.
100,000
MOMENT
(IN-LBS.)
-100,000
-200,000
I= 120 000
-300,000
.1
DEFLECTION
(IN)
-.1
-.2
-.3
-.4
-.5
-.6
I= 15000
CENTER LIFT I



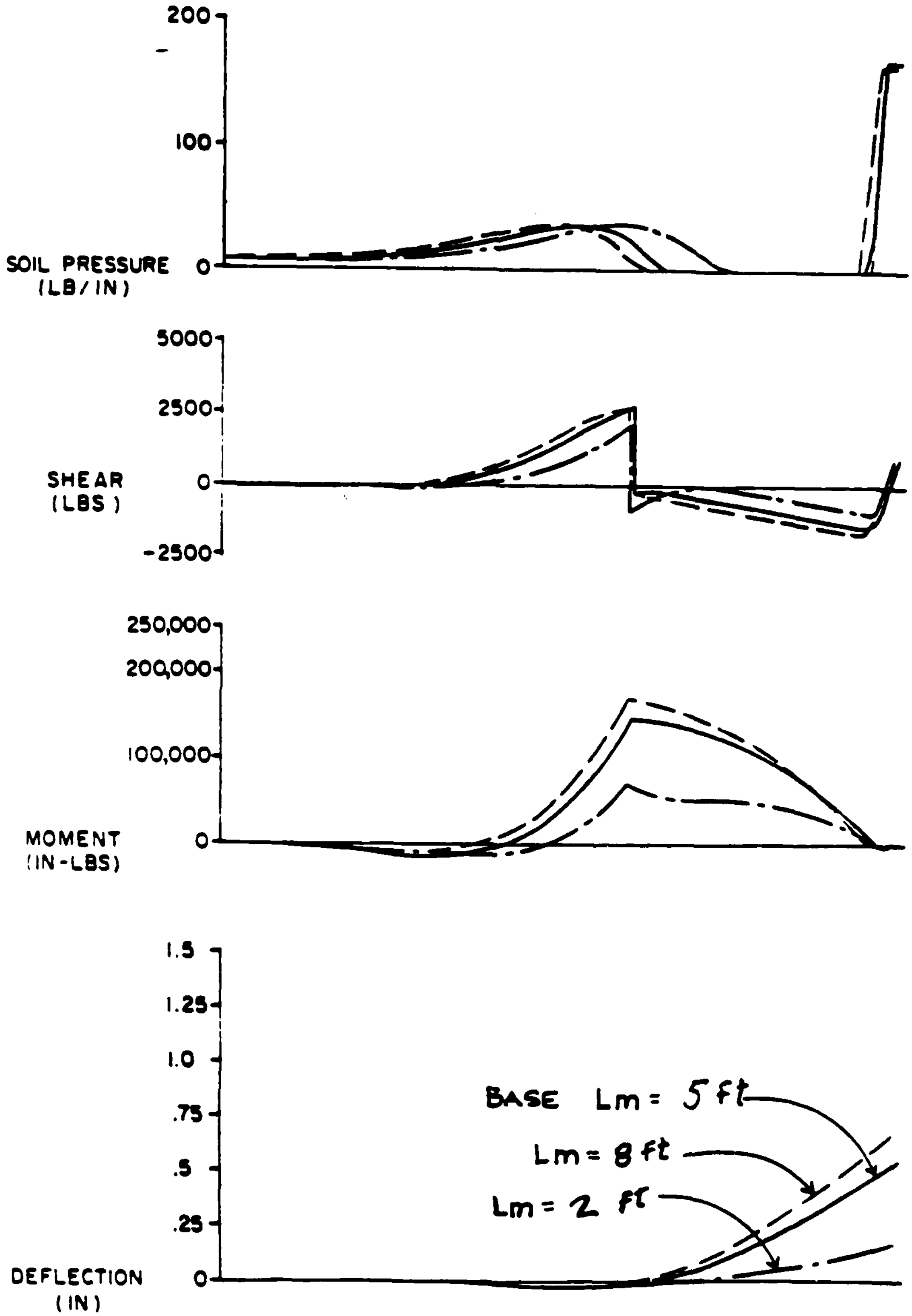
200.
100.
BASE SPACING = 20FT
12FT
16FT
24FT
SOIL PRESSURE
(LB./IN.)
5000
2500
SHEAR
(LBS)
-2500
-5000
100,000
MOMENT
(IN-LBS.)
-100,000
-200,000
-300,000
.1
DEFLECTION
(IN.)
-.1
-.2
-.3
-.4
-.5
-.6
S = 12FT
S = 24FT
CENTER LIFT S



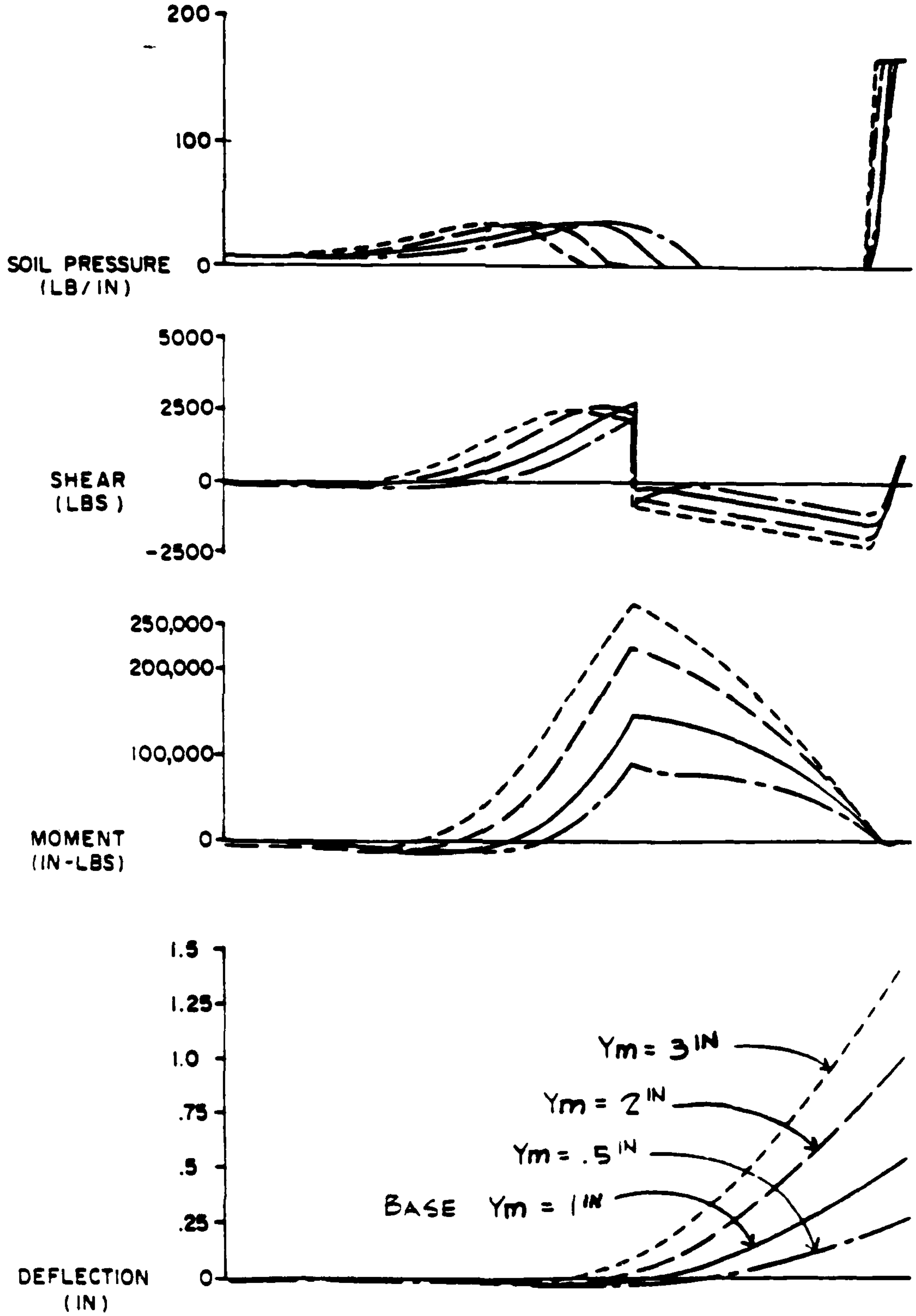
200.
100.
$P_p = 5^k/_{FT}$
BASE $P_p = 3^k/_{FT}$
$P_p = 1^k/_{FT}$
SOIL PRESSURE (LB./IN.)
5000
2500
SHEAR (LBS.)
-2500
-5000
100,000
MOMENT (IN-LBS.)
-100,000
-200,000
-300,000
.1
DEFLECTION (IN.)
-.1
-.2
-.3
-.4
-.5
-.6
CENTER LIFT P_p



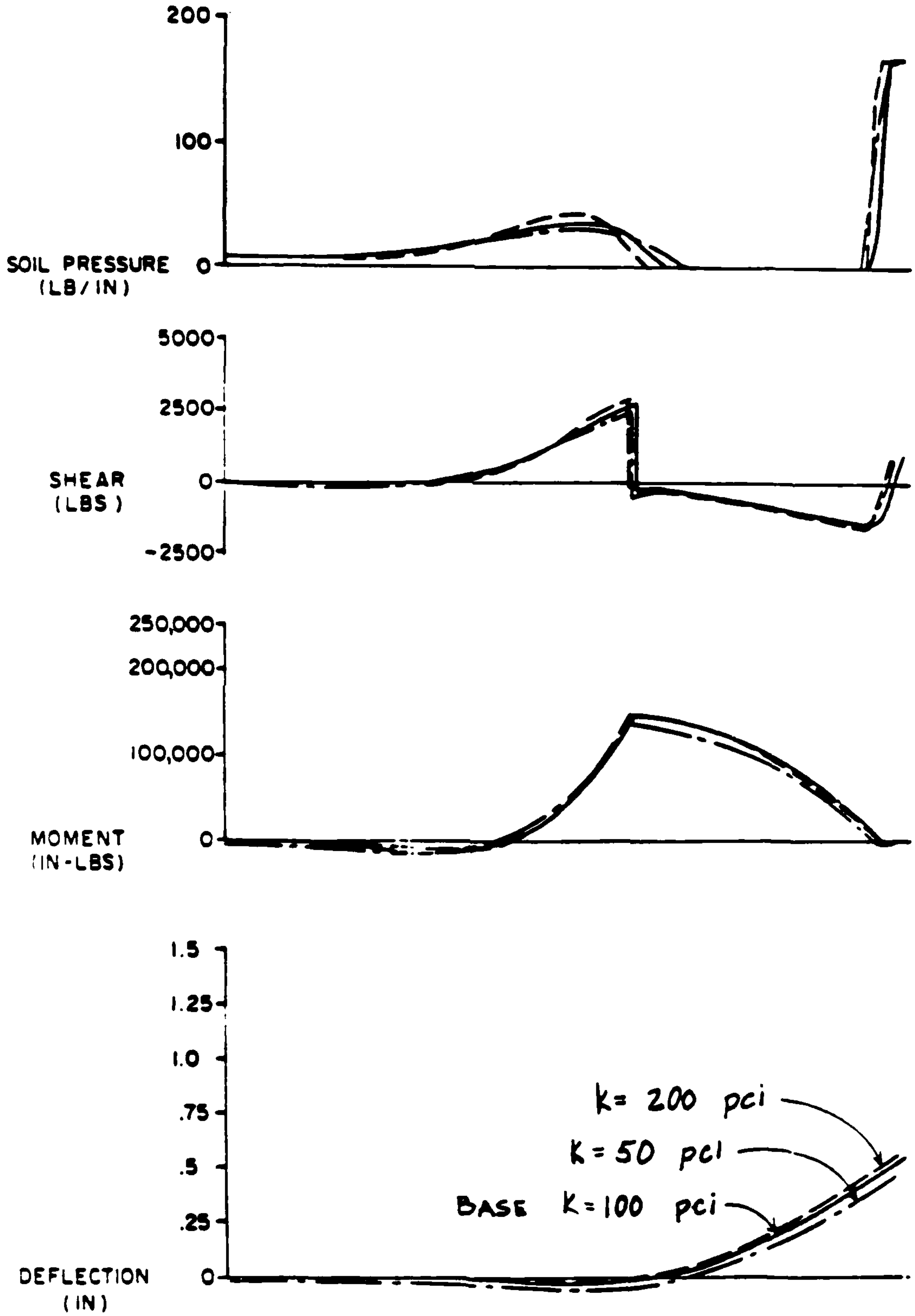
CENTER LIFT P_i


200
100
SOIL PRESSURE (LB/IN)
0
5000
2500
SHEAR (LBS)
0
-2500
250,000
200,000
100,000
MOMENT (IN-LBS)
0
1.5
1.25
1.0
.75
.5
.25
DEFLECTION (IN)
0
BASE Lm = 5 ft
Lm = 8 ft
Lm = 2 ft

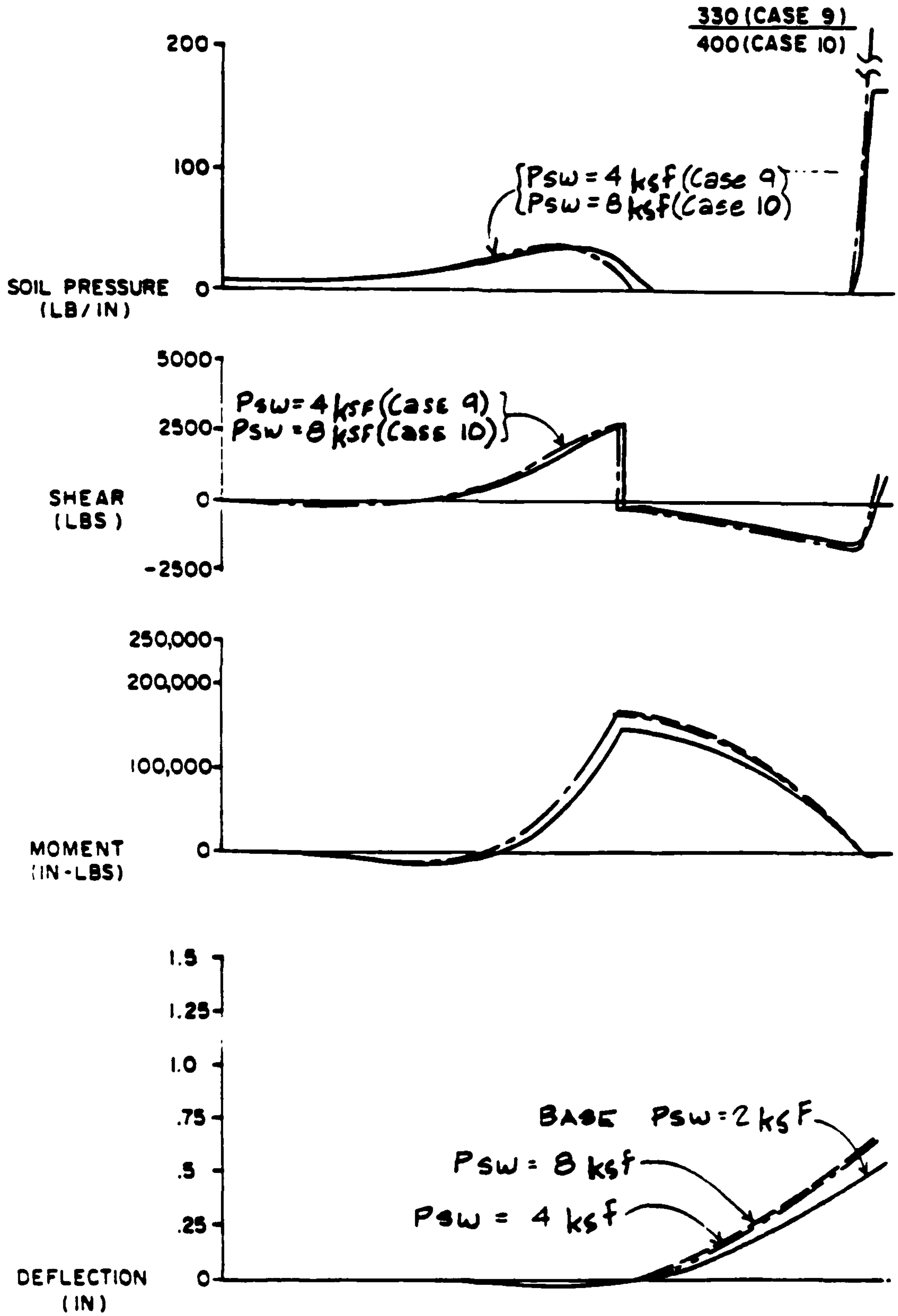
EDGE LIFT Lm


200
100
SOIL PRESSURE
(LB/IN)
0
5000
2500
SHEAR
(LBS)
0
-2500
250,000
200,000
100,000
MOMENT
(IN-LBS)
0
1.5
1.25
1.0
.75
.5
.25
DEFLECTION
(IN)
0
Ym = 3 IN
Ym = 2 IN
Ym = .5 IN
BASE Ym = 1 IN

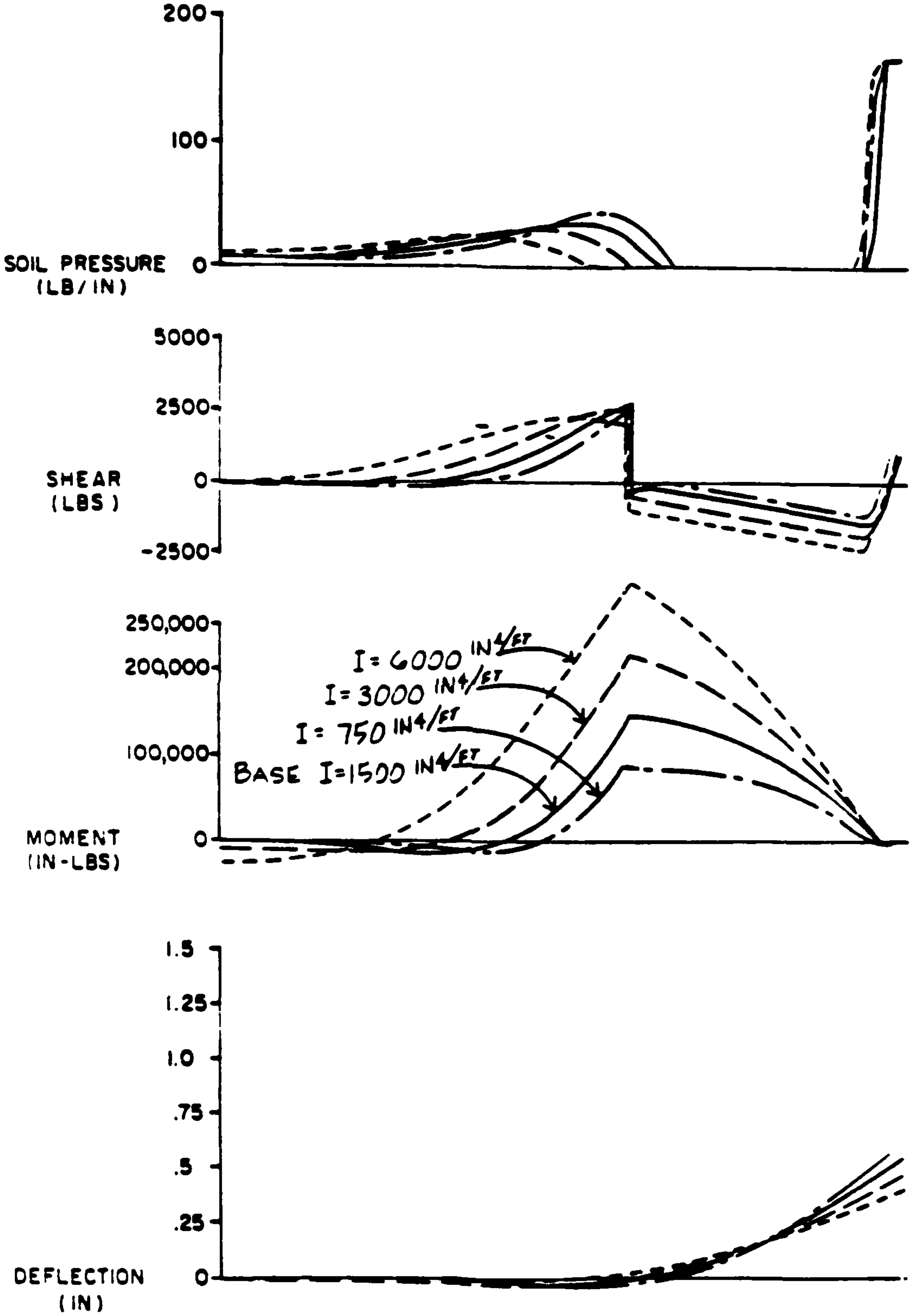
EDGE LIFT Ym


200
100
SOIL PRESSURE (LB/IN)
0
5000
2500
SHEAR (LBS)
0
-2500
250,000
200,000
100,000
MOMENT (IN-LBS)
0
1.5
1.25
1.0
.75
.5
.25
DEFLECTION (IN)
0
k= 200 pci
k= 50 pci
BASE k= 100 pci

EDGE LIFT k

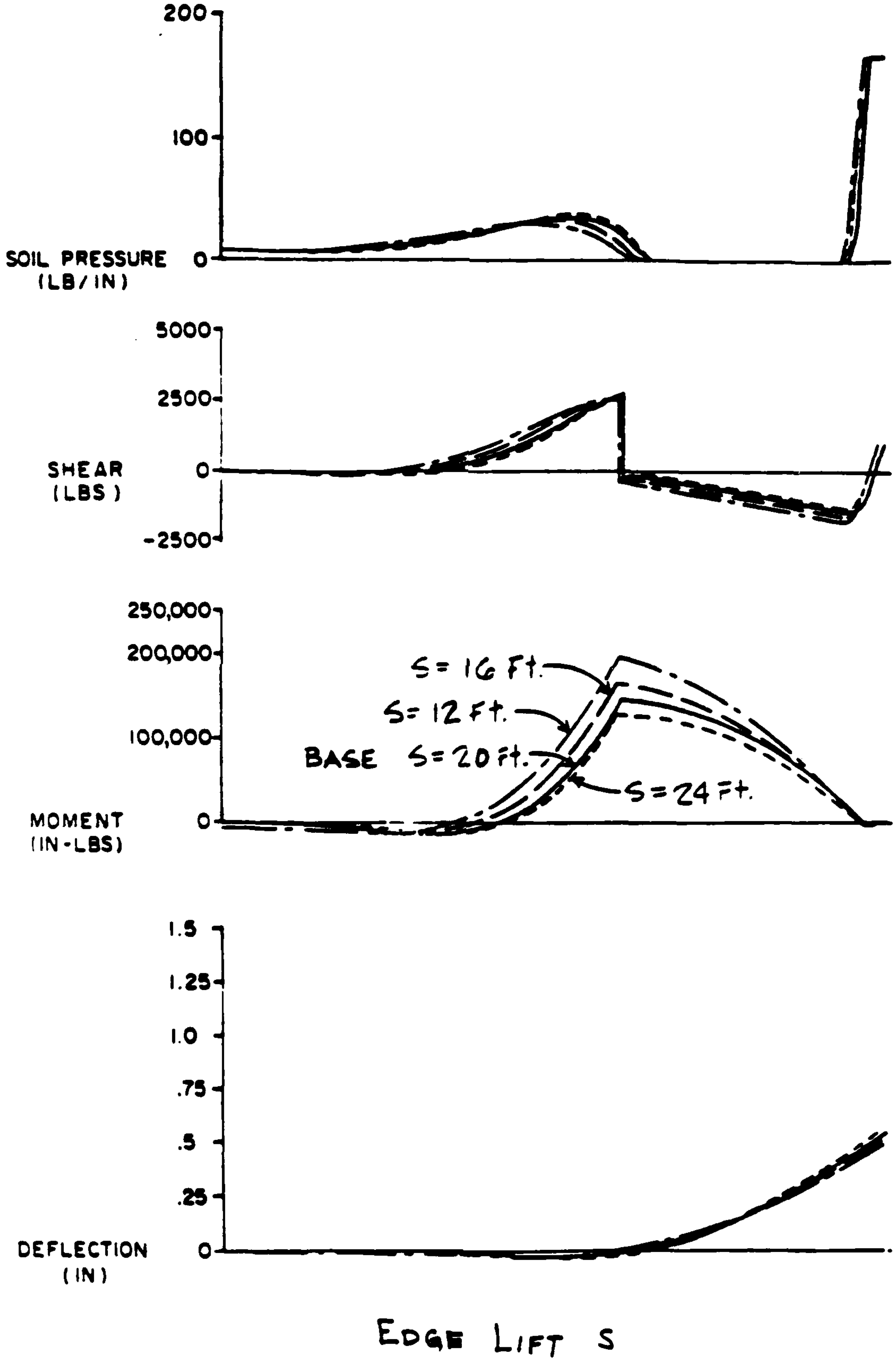


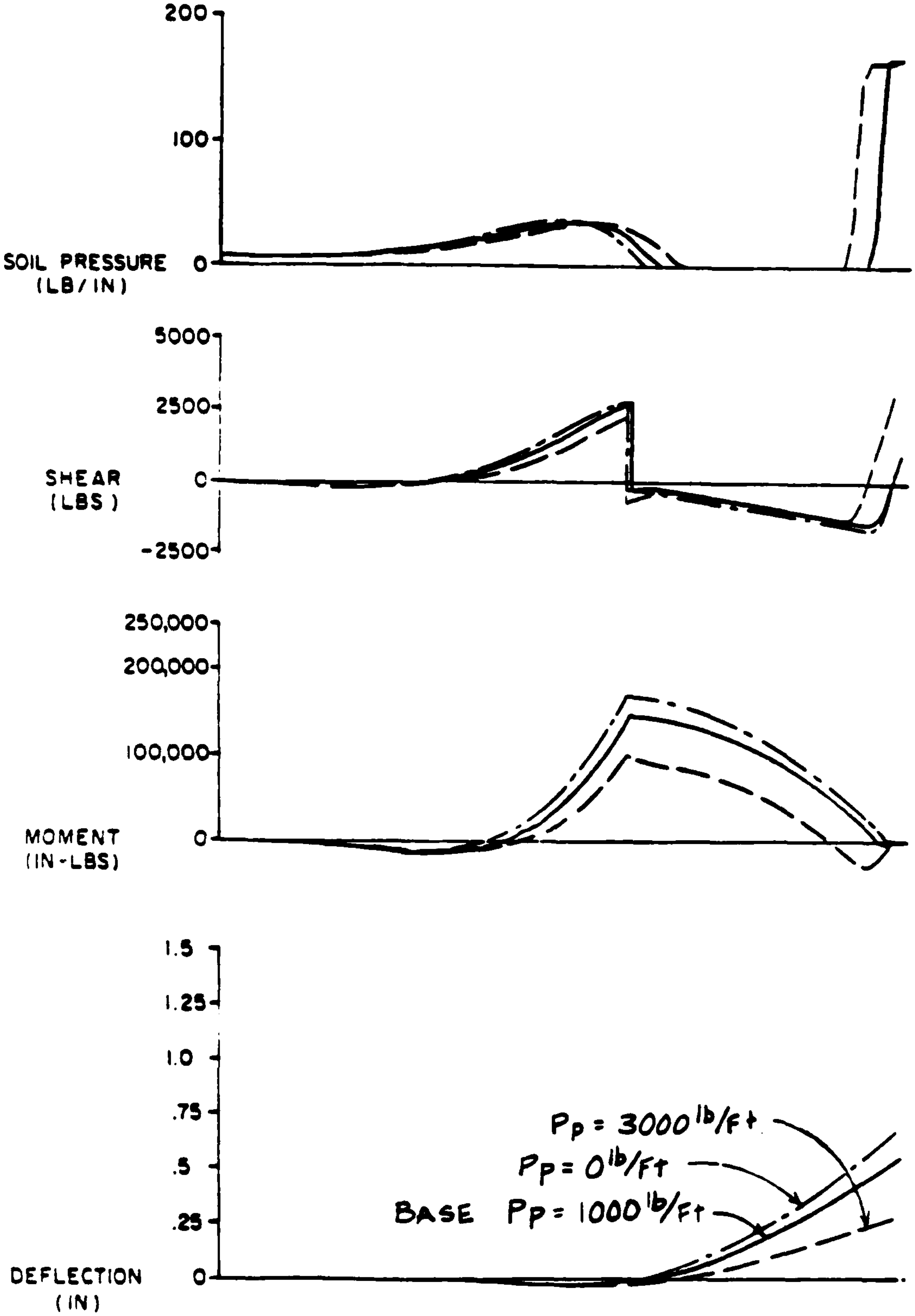
EDGE LIFT Psw



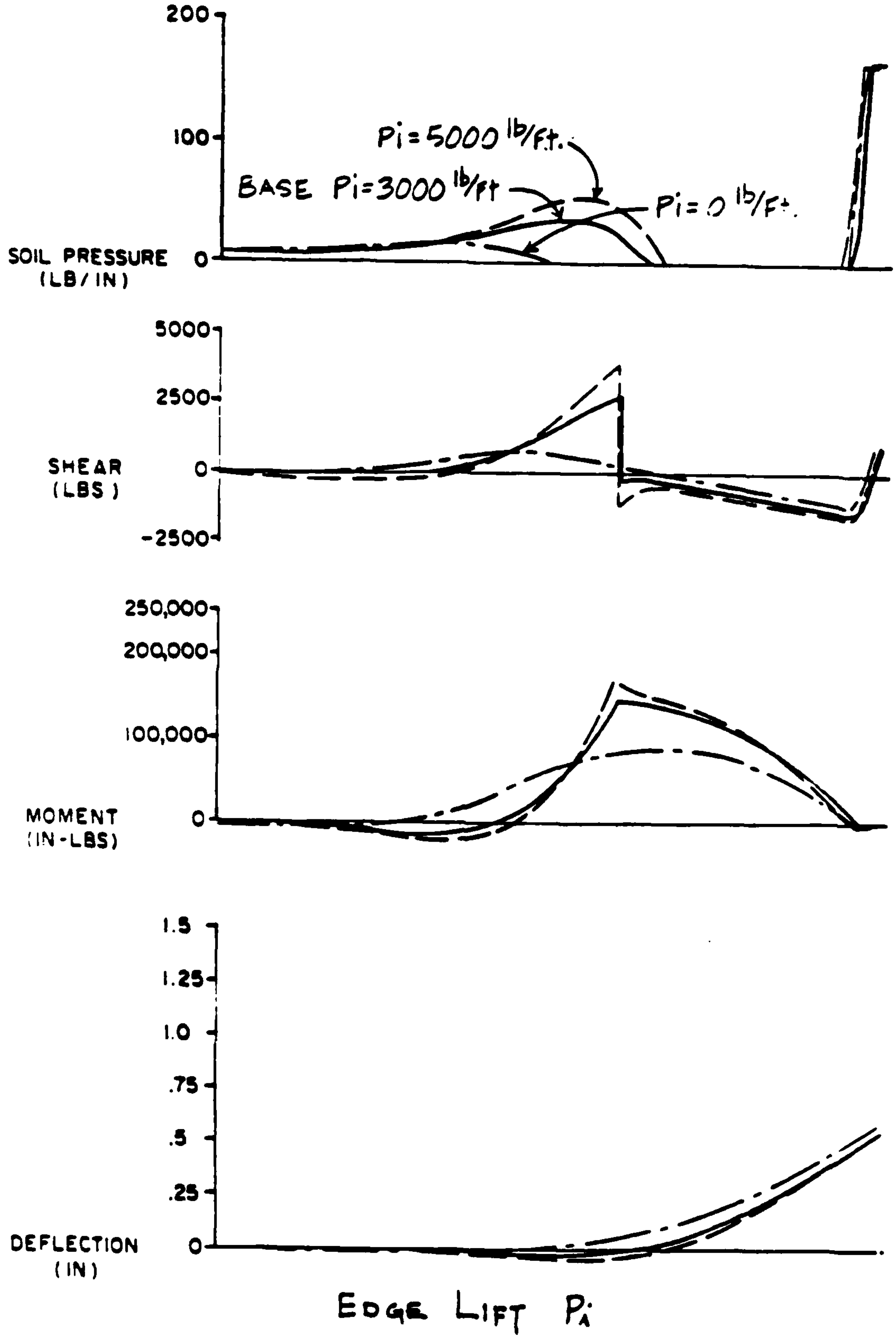
200
100
0
SOIL PRESSURE (LB/IN)
5000
2500
0
-2500
SHEAR (LBS)
250,000
200,000
100,000
0
MOMENT (IN-LBS)
I = 6000 IN4/FT
I = 3000 IN4/FT
I = 750 IN4/FT
BASE I = 1500 IN4/FT
1.5
1.25
1.0
.75
.5
.25
0
DEFLECTION (IN)

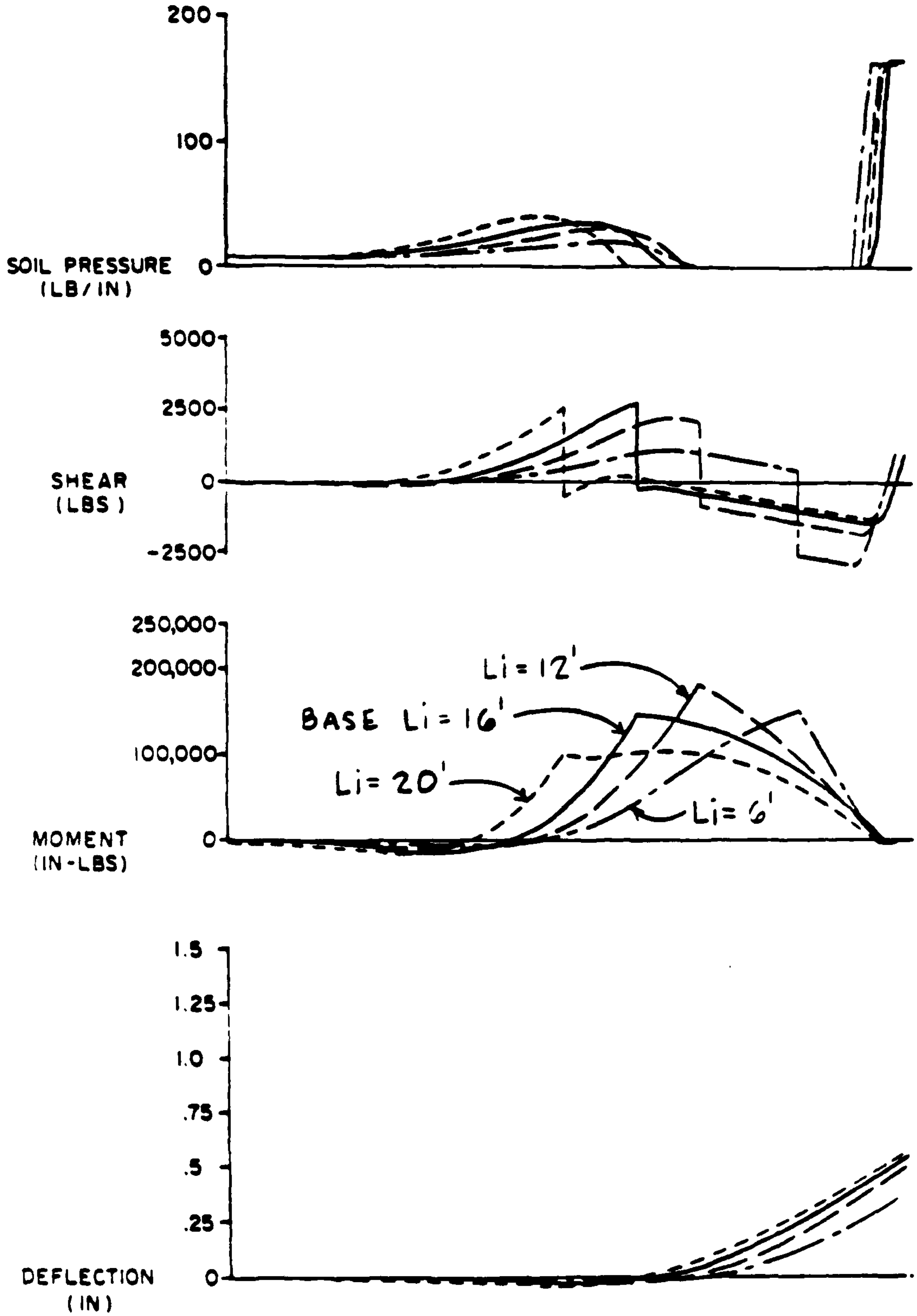
EDGE LIFT I


200
100
SOIL PRESSURE (LB/IN)
0
5000
2500
SHEAR (LBS)
0
-2500
250,000
200,000
S= 16 Ft.
S= 12 Ft.
100,000
BASE S= 20 Ft.
S= 24 Ft.
MOMENT (IN-LBS)
0
1.5
1.25
1.0
.75
.5
.25
DEFLECTION (IN)
0
EDGE LIFT S


200
100
SOIL PRESSURE (LB/IN) 0
5000
2500
SHEAR (LBS) 0
-2500
250,000
200,000
100,000
MOMENT (IN-LBS) 0
1.5
1.25
1.0
.75
.5
.25
DEFLECTION (IN) 0
Pp = 3000 lb/Ft
Pp = 0 lb/Ft
BASE Pp = 1000 lb/Ft

EDGE LIFT Pp


200
100
Pi=5000 lb/ft.
BASE Pi=3000 lb/ft
Pi=0 lb/ft.
SOIL PRESSURE
(LB/IN)
0
5000
2500
SHEAR
(LBS)
0
-2500
250,000
200,000
100,000
MOMENT
(IN-LBS)
0
1.5
1.25
1.0
.75
.5
.25
DEFLECTION
(IN)
0
EDGE LIFT Pi


200
100
SOIL PRESSURE 0
(LB/IN)
5000
2500
SHEAR 0
(LBS)
-2500
250,000
200,000
100,000
MOMENT 0
(IN-LBS)
Li = 12'
BASE Li = 16'
Li = 20'
Li = 6'
1.5
1.25
1.0
.75
.5
.25
DEFLECTION 0
(IN)

EDGE LIFT Li

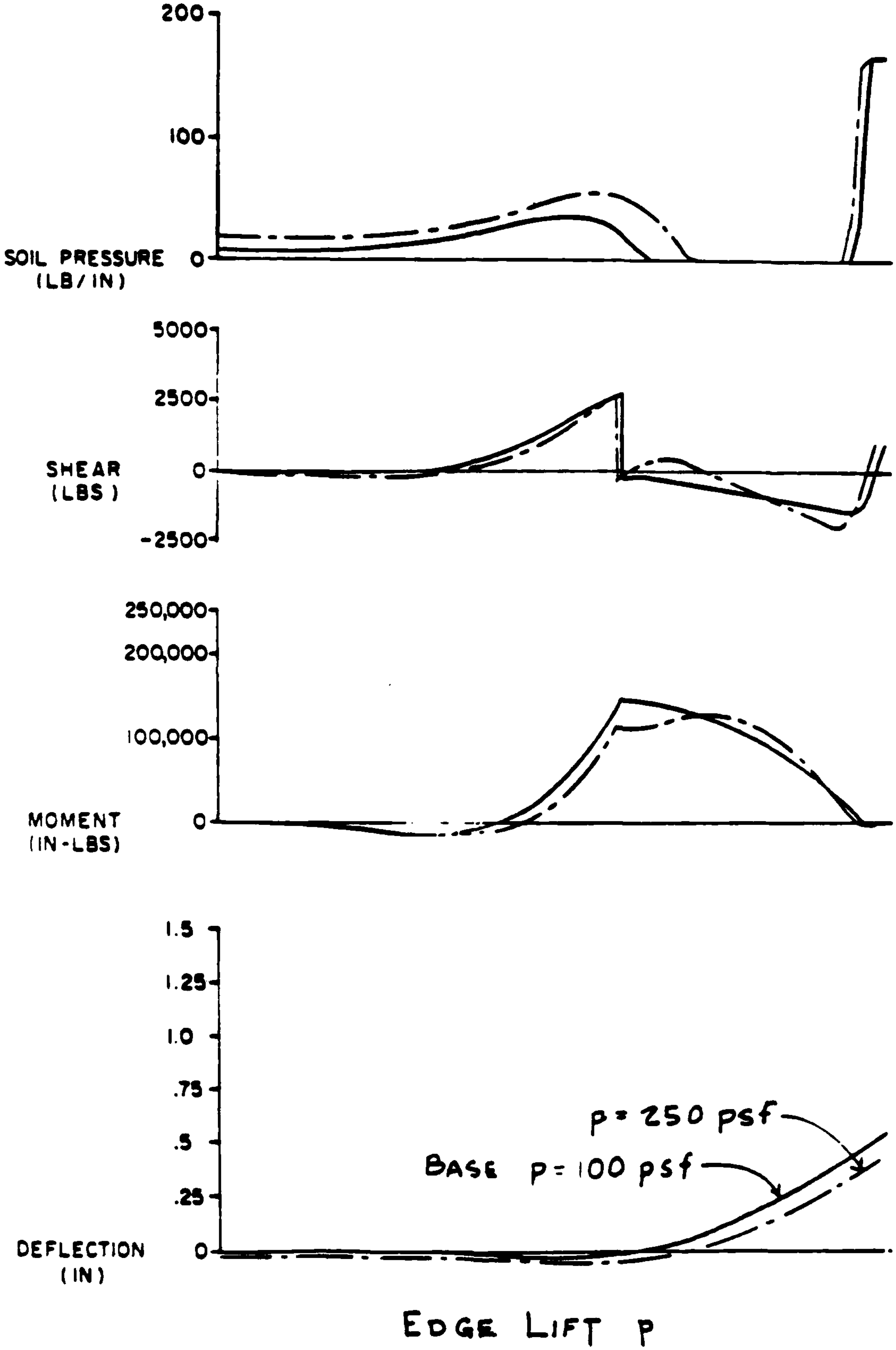

200
100
SOIL PRESSURE (LB/IN)
0
5000
2500
SHEAR (LBS)
0
-2500
250,000
200,000
100,000
MOMENT (IN-LBS)
0
1.5
1.25
1.0
.75
.5
.25
DEFLECTION (IN)
0
p = 250 psf
BASE p = 100 psf
EDGE LIFT P

Exhibit 1

Design Criteria for Ribbed Mat Foundations

DESIGN OF RIBBED MAT FOUNDATIONS

BY

JOSEPH P. HARTMAN

AND

B. H. JAMES

U.S. ARMY CORPS OF ENGINEERS

SOUTHWESTERN DIVISION

DALLAS, TEXAS

REVISED

DECEMBER 1986

CONTENTS

PART I - GENERAL REQUIREMENTS FOR RIBBED MATS

1. REFERENCES.

1.1 Engineering Instruction Manual, Corps of Engineers, Southwestern Division, (latest edition).

1.2 "Criteria for Selection and Design of Residential Slabs-on-Ground," Building Research Advisory Board (BRAB).

1.3 "Design and Construction of Post-Tensioned Slabs-on-Ground," Post Tensioning Institute (PTI) 1980.

1.4 TM 5-818-7, Foundations in Expansive Soils, Corps of Engineers, 1983.

2. BACKGROUND. Ribbed mat foundations consist of a thin slab on grade which acts monolithically with a grid of stiffening beams beneath the slab. The beams (ribs) are cast in trenches dug in the foundation soil. Ribbed mats combine the economic advantages of shallow foundations with the performance advantages of monolithic floors. Ribbed mats are especially useful for minimizing differential foundation movements in areas with expansive soils.

3. DESIGN METHODS.

3.1 EXPANSIVE SOILS.

3.1.1 Behavior.

3.1.1.1 Center Lift. In the center lift condition the soil near the edge of the slab drops in relation to the soil near the center. This is due to moisture retention by the interior soils and the drying and shrinking of perimeter soils. As this occurs, the perimeter soil provides less support for the edge of the slab which then acts as a cantilever. This is illustrated in Figure A1 of Appendix A.

3.1.1.2 Edge Lift. In the edge lift condition the soil near the edge of the slab rises in relation to the soil near the center. This is due to the increasing moisture content and subsequent swelling of soil near the edge. The swelling soil raises the edge of the slab, causing some of the slab to lift off the soil. Interior loads cause the slab to sag and recontact the soil at some interior location. The slab thus tends to act as a beam, simply supported by the soil at the edge, and by soil support near the center of the slab. The amount of support at the center depends on numerous parameters such as interior loads, rib bending stiffness, soil swell pressures, and the magnitude of soil swelling. Typical edge lift behavior is illustrated in Figure A3 of Appendix A.

3.1.2 SWD Method. All ribbed mats on expansive soils shall be designed in accordance with the provisions of Part II of this report. However, ribbed mats for family housing may be designed in accordance with paragraphs 3.1.3 or 3.1.4.

3.1.3 PTI Method. The PTI method may only be used for design of family housing foundations on expansive soils. Specifically, slab width (short dimension) should not exceed 40 feet, rib depths should not exceed 24 inches, loading should consist only of perimeter loads and light interior distributed loads ($DL+LL \leq 100$ psf), soils should be fairly weak in-situ materials with no extensive substitution of non-expansive fill. When calculating deflections for a conventionally reinforced slab, use the cracked moment of inertia with the PTI formulas. Section properties for calculation of bending stresses shall consider an effective flange for each rib, as limited by ACI 318-83, sections 8.10.1 and 8.10.3.

3.1.4 BRAB Method. The BRAB report may only be used for design of foundations for family housing. However, the PTI method is preferred, since the BRAB method may produce unreasonable results for large foundations.

3.1.5 Computer Method. In lieu of paragraph 3.1.2, ribbed mats may be designed using appropriate computer programs. Such programs must be capable of modeling the variable soil swell due to moisture changes, and the non-linear soil-structure interaction near the perimeter of the foundation. One such computer program is CBEAMC, program X0050 in the Corps of Engineers Civil Engineering Library.

3.1.6 Load Factors. When using the above methods to design ribbed mats for center lift and edge lift conditions, load factors may be multiplied by .75 (strength method) or allowable stresses may be increased by one-third (working stress method).

3.2 NON-EXPANSIVE SOILS. Ribbed mat slabs on non-expansive soils need not be designed for bending due to center lift or edge lift conditions. Beam on elestic foundation analyses may be used to determine the effects of concentrated loads on ribs, or ribs may be designed as conventional strip or spot footings.

3.3 SOIL PROPERTIES. Soil properties for design of ribbed mats will be as provided in the Foundation Design Analysis by the Corps of Engineers. Properties necessary for design in accordance with paragraph 3.1.2 consist of the following, which are defined in Appendix A:

- qa - allowable bearing pressure
- k - subgrade modulus
- Ym - soil heave
- Lm - edge moisture variation distance
- Psw - maximum pressure of swelling soil

4. MINIMUM REQUIREMENTS.

4.1 SUBGRADE PREPARATION. A vapor barrier, capillary water barrier, and a minimum of 18 inches of non-expansive fill will normally be used beneath ribbed mats. Additional non-expansive fill will often be used to lessen the effects of highly expansive soils. These requirements will be detailed in the Foundation Design Analysis.

4.2 SLAB. For family housing and other small lightly loaded buildings a 4 inch slab may be used. For other buildings the minimum slab thickness will be 5 inches. Minimum slab reinforcing shall be 0.2 percent. Where slabs are subjected to vehicular loading they must be designed for the maximum wheel load, similar to paving. Use 650 psi flexural strength concrete for slabs subject to wheel loads.

4.3 GRID GEOMETRY. Ribs should be located to form a continuous grid. Rib spacing should not exceed 20 feet in expansive soils, or 25 feet in non-expansive soils. Locations of ribs should conform to significant wall and column loads, and may be used to resist thrusts from rigid frame reactions. Ribs should be provided around large openings in the slab. In expansive soils diagonal ribs are required at exterior corners.

Expansion joints should be provided at 250 foot intervals, and should also be used to break irregularly shaped buildings into rectangular segments. Foundations for family housing do not require expansion joints due to irregular shapes.

4.4 RIB SIZE. Minimum rib depth is 20 inches. Rib depths should usually not exceed 3 feet to minimize construction difficulties related to placing reinforcement and maintaining trench walls. If deeper ribs are used, rib width should also be increased. Minimum rib width is 12 inches except for family housing foundations, where 10 inch ribs may be used. Sufficient rib width must also be provided to transfer wall and column loads to the soil as strip footings. The allowable soil bearing capacity may not be exceeded when considering the width of the rib plus an effective slab width on each side of the rib. The effective slab width for bearing is limited to the thickness of the slab. At column locations an alternate is to provide fillets at rib intersections, sufficient to act as spot footings for column loads.

4.5 RIB CAPACITY. Concrete should have a minimum compressive strength of fc=3000 psi at 28 days. Reinforcing shall be grade 60, except ties may be grade 40. Minimum reinforcing ratio (As/Ag) shall be .0033 top and .0033 bottom, this may be reduced to .005 total in non-expansive soils. Use #3 ties at 24 inches. minimum. These minimums should be sufficient for shrinkage stresses and for unpredictable soil behavior.

4.6 PRESTRESSED MATS. For prestressed ribbed mats all the above minimum requirements apply except that slab and rib top reinforcement may be deleted and replaced by appropriate post-tensioning strands. Mild steel shall still be provided in the bottom of ribs. Minimum prestress shall be 100 psi on the gross area, including effects of subgrade friction as calculated by the PTI method, reference 1.3. Concrete tensile stress shall be limited to $3\sqrt{f'c}$ and shear stress limited to $1.1\sqrt{f'c}$. A one-third overstress may be allowed per paragraph 3.1.6.

4.7 CONSTRUCTION DETAILS.

4.7.1 Conventionally Reinforced. Construction joint spacing should not exceed 50 feet in either direction. A horizontal construction joint may be provided in the ribs at the base of the capillary water barrier when unstable trench walls may cause construction difficulties.

4.7.2 Prestressed. Construction joint spacing shall not exceed 75 feet in either direction. Tendons within each placement shall be stressed to 15 percent of the final prestress not more than 24 hours after the concrete has attained sufficient strength to withstand the partial prestress. Other construction procedures for prestressed ribbed mats shall conform to reference 1.3.

4.7.3 Contractor Designs. Ribbed mat foundations may be designed as prestressed or conventionally reinforced as selected by the engineer. The plans and specifications shall not include the option of changing the ribbed mat from one type to another. The reason for this prohibition is that design parameters (e.g., moments of inertia) may be dependent on the type of ribbed mat being designed and may affect calculated shears and moments. This does not prohibit revisions of the slab type as a result of contractor value engineering proposals. However, such revisions must include a complete design of the ribbed mat foundation using appropriate design parameters in accordance with this report.

PART II - ANALYSIS OF RIBBED MAT FOUNDATIONS ON EXPANSIVE SOILS

1. SCOPE. This part of the report contains the basic rules for design of ribbed mats in expansive soils. This method may be used to predict shears, moments and deflections in ribs subject to soil movement due to changing moisture content. For a commentary on the design method refer to Appendix A; for example design calculations refer to Appendix B. The design method from Part II should be used in conjunction with the "minimum requirements" for ribbed mats, as presented in Part I.

2. GENERAL

2.1 NOTATION.

- C = Correction factor for equivalent cantilever length
- D = Beam deflection (IN)
- I = Moment of inertia per foot, I=Ir/S (IN^4/FT)
- Ir = Moment of inertia of rib (IN^4)
- k = Modulus of subgrade reaction (PCI)
- Lo = Basic length of cantilever (FT)
- Lc = Equivalent length of cantilever, center lift (FT)
- Le = Equivalent length of simple beam, edge lift (FT)
- Li = Distance from perimeter to location of interior load (FT)
- Lm = Edge moisture variation distance (FT)
- Lb = Width of soil bearing at perimeter, edge lift (FT)
- M = Bending moment per foot (FT-KIP/FT)
- Mr = Bending moment per rib, Mr=MxS (FT-LB)
- Pi = Interior load (PLF)
- Pp = Perimeter load (PLF)
- Psw = maximum pressure of swelling soil (PSF)
- R = End reaction at perimeter for equivalent simple beam (LB)
- S = Rib spacing (FT)
- w = Uniform load (PSF)
- V = Shear per foot (LB/FT)
- Vr = Shear per rib, Vr=VxS (LB)
- Ym = Soil heave (IN)
- θ = Rotation of support of equivalent cantilever (RAD)

2.2 UNITS. The equations presented in section 3 are written for units as defined in the above notation. If other units are used the equations must be modified appropriately.

2.3 RIB DEFINITIONS. Ribs are defined as perimeter, transverse or diagonal as shown in Figure 1. Note that transverse refers to ribs parallel to either axis of the building.

FIGURE 1 - RIB DEFINITIONS

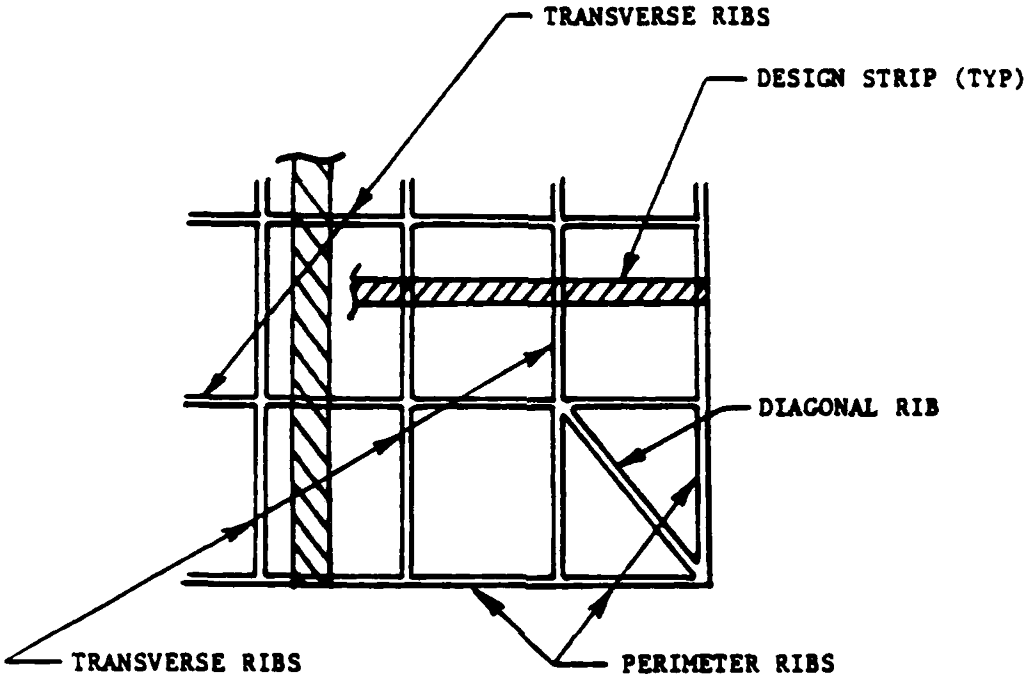
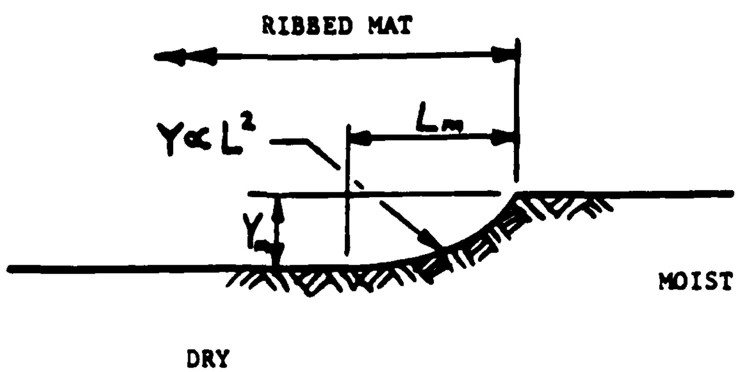


FIGURE 2 - SOIL EDGE PROFILE



2.4 STRIP ANALYSIS. The analysis is based on a strip assumption, ignoring the effects of the grid configuration of the ribs. The formulas and examples presented below are for an equiv ilent 1-foot strip, using "per foot" values for loads and stiffness.

2.5 SOIL EDGE PROFILE. For edge lift the maximum swell occurs at the perimeter and decreases rapidly toward the interior. The soil profile is assumed to be parabolic (in the unloaded condition) and is illustrated in Figure 2.

3. ANALYSIS METHOD.

3.1 TRANSVERSE RIB - CENTER LIFT.

3.1.1 General. Center lift analysis is based on an equivalent cantilever beam to determine moments, shears and deflections.

3.1.2 Moment. The length of the equivalent cantilever can be calculated as:

$$Lc = C \times Lo$$

where: $Lo = 2.3 + .4\ Lm$

$$C = .8\ Ym^{.12}\ I^{.16} / Pp^{.12}$$

The maximum moment may then be calculated from statics using conv tional cantilever formulas such as:

$$M = Pp\ Lc + 1/2\ w\ Lc^2$$

The moment can then be assumed to be constant for a distance Lc/2 and then to decrease linearly from M at the cantilever support, to near zero at a distance 5Lc from the perimeter. To obtain the design moment for a given rib, multiply the calculated per-foot moment by the appropriate rib spacing ($Mr = M \times S$).

3.1.3 Shear. The maximum shear may be calculated from statics using the same equivalent cantilever as for moment.

$$V = Pp + w\ Lc$$

The shear may then be assumed to decrease linearly from V at the cantilever support, to near-zero at a distance 5Lc from the perimeter. To obtain the design shear for a given rib, multiply the calculated per-foot shears by the appropriate rib spacing ($Vr = V \times S$).

3.1.4 Deflection. Deflection at the perimeter is the sum of three components: bending deflection of the equivalent cantilever, vertical translation of the cantilever support, and rotation of the cantilever support. Rotation of the support may be calculated as:

$$\theta = M^{1.4} / 9800\ I\ k^{.5}$$

The perimeter deflection is then:

$$D = .11 + 12\ Lc\ \theta$$

where .11 inches is an approximation for the support translation plus the cantilever bending, and (12 Lc) is the length in inches.

Use the deflection calculated above to compare with allowable deflection. The allowable deflection may be determined by using 4Lc as the length between points of zero and maximum deflection.

3.2 TRANSVERSE RIB - EDGE LIFT.

3.2.1 General. Edge lift analysis is based on an equivalent simple beam, supported at the perimeter and at some interior location.

3.2.2 Deflection. The first step in calculating deflection is to determine the length of the equivalent simple beam. The appropriate length depends on many parameters, including the deflection. Therefore, deflection must first be estimated to determine equivalent length, then a deflection is calculated based on that length. The process is repeated until calculated deflection matches the assumed deflection. The equivalent simple beam length may be calculated as:

$$Le = 7.5\ I^{.17}\ Li^{.37}\ D^{.12} / w^{.07}\ Pi^{.11}$$

The perimeter end reaction (R) for this beam may be calculated from statics. For an ideal case the reaction is:

$$R = Pp + 1/2\ w\ Le + Pi(Le-Li)/Le$$

The width of soil bearing at the perimeter can be approximnated as:

$$Lb = 1.1\ (R/Psw)$$

The edge deflection is found by determining the soil swell at a distance Lb from the perimeter, based on the parabolic swell profile:

$$D = Ym(Lm-Lb)^2/Lm^2$$

When satisfying deflection criteria, use the calculated deflection and equivalent simple beam length.

3.2.3 Moment. Once the simple beam equivalent length has been determined, the bending moments may be calculated based on statics.To obtain rib design moments, multiply per-foot moments by the rib spacing.

3.2.4 Shear. Once the simple beam equivalent length has been determined, the shears may be calculated based on statics. To obtain rib design shears, multiply per-foot shears by the rib spacing. Near the interior support the design shear need not exceed:

$$V = Pi + w(Le-Li)$$

This is due to the effects of distributed soil support.

3.2.5 Special Cases. If Pi=0 or if Li>Le substitute the value

$$1.4 = Li^{.37} / Pi^{.11}$$

The equation for the simple beam length would then become:

$$Le = 10.5\ I^{.17}\ D^{.12} / w^{.07}$$

3.3 PERIMETER RIB.

3.3.1 Center Lift. For center lift the perimeter rib will have no support from the soil and must be designed to span between transverse ribs for the calculated perimeter load (P_D).

3.3.2 Edge Lift. For edge lift the soil pressure on t perimeter rib will exceed the applied perimeter loads. The perimeter rib must designed to span between transverse ribs for this net upward force.

3.4 DIAGONAL RIB. Diagonal ribs are used to support exterior corner for center lift conditions, if loss of support occurs under both perimeter ribs. Diagonal ribs must be designed to provide the same moment and shear capacity as the larger of the two adjacent transverse ribs.

3.5 INTERIOR RIB. Interior ribs and rib intersections should be located at significant wall and column loads. The ribs should be designed for these loads as strip or spot footings, using beam-on-elastic-foundation methods. Differential soil movement due to moisture change is assumed not to occur except at the perimeter. However, to account for unpredictable interior soil movements, interior ribs must have the minimum size and capacity as required in Part I.

APPENDIX A - COMMENTARY ON PART II

1. SCOPE. Actual behavior of ribbed mats in expansive soils involves complex, non-linear, soil-structure interaction. The best solution for such behavior is provided by computer programs. The hand design method has been developed to approximate such computer results. Hand solutions have been checked by computer analyses; results have been within acceptable limits of error. However, such checks have been made only for a limited range for each design parameter, as shown in Table A1, corresponding to the usual values for military construction within Southwestern Division. If a wider range of parameters is applied to the hand design formulas, the results may be less accurate.

TABLE A1

Parameter	Units	Minimum	Maximum
k	pci	50	200
Ym	in	0.5	3.0
Lm	ft	2	8
I	in^4/ft	750	6000
Pp	lb/ft	1000	5000
Pi	lb/ft	0	5000
Li	ft	6	20
w	psf	100	250
Psw	psf	2000	8000

2. GENERAL.

2.1 NOTATION.

Ir = moment of inertia of rib. For non-prestressed rib mats Ir should be the effective moment of inertia, calculated per ACI 318, Section 9.5.2.3.

k = Modulus of subgrade reaction. This parameter is the ratio of the soil pressure at the base of the concrete and the corresponding settlement. Since modulus values are typically determined by plate-load test at the ground surface, they should be corrected for depth and for footing size (expected high pressure area between concrete and soil). Analyses have indicated that the high bearing pressure area for center lift conditions will occur in an area several feet long parallel to the transverse rib and several feet on each side of the rib. A crude approximation for this area would be 5 feet square. This approximation should be adequate for design since calcualations are not sensitive to the modulus of subgrade reaction.

qa = Allowable bearing pressure. This is the safe bearing capacity of the soil at the base of the ribs. A factor of safety of 3.0 is recommended for computing this value.

Lm = Edge moisture variation distance. This represents the distance, inward from the edge of the slab, over which the moisture content of the soil changes. Much judgement is required in determining this value.

Psw = Maximum pressure of swelling soil. This is the maximum potential pressure between the soil and the base of the exterior rib, due to an increase in soil moisture content, if the rib is prevented from rising. Calculation of Psw should include consideration of pressure distributions in the underlying soils.

Ym = Soil heave. This is the differential vertical movement of the soil representing either soil heave (edge lift) or soil shrinkage (center lift). The magnitude of Ym is the computed vertical movement of a particle of soil at the ground surface due to a change in moisture content. This value should be based on the accumulation of potential volume changes for the full thickness of the active zone (Za), with no significant loads applied to the foundation. The value of Ym may differ for edge lift and center lift conditions.

Pi, Pp, w = Applied loads. Loads should consist of full dead plus live loads; including dead load of the slab and ribs.

2.2 UNITS.

2.3 RIB DEFINITIONS.

2.4 STRIP ANALYSIS. The hand solution formulas have been developed for analysis of an equivalent 1 foot strip. This is convenient for uniform loads and for soil properties, but requires some calculations for appropriate concentrated loads and bending stiffness. Rib stiffness must be divided by rib spacing to get the per-foot stiffness. If column loads exist they must also be divided by the rib or column spacing to provide an equivalent load per foot. If interior wall loads are parallel to the transverse rib, they must be divided by the rib spacing. These calculations are illustrated in Appendix B.

2.5 SOIL EDGE PROFILE. The edge lift condition occurs when increased moisture content swells exterior soils, and this effect extends under the edge of the slab. The center lift condition occurs when soils under the slab are generally moist and seasonal drying occurs on the exterior, again extending under the edge of the slab. This causes the soil at the edge to shrink away from the slab.

The analysis method is based on an assumed parabolic swell profile which occurs uniformly along the perimeter. This is a convenient idealization of real soil behavior, which must be more erratic. However, the parabolic profile has better correlation with measured swells than do other possible edge profile assumptions. Note that the soil profile is not used in the hand design formulas for center lift. However, a parabolic profile was used in the computer analyses for center lift, which formed the basis for the hand design forumlas.

3. ANALYSIS METHOD. Many of the formulas for shears, moments and reactions are idealized, assuming Pp and R are exactly at the perimeter and that w extends to the perimeter. These approximations should usually be acceptable, but the formulas may be modified to account for actual load patterns.

3.1 TRANSVERSE RIB - CENTER LIFT

3.1.1 General. Typical behavior of a transverse rib for center lift conditions is shown in Figure A1. This illustrates the soil bearing pressure and the shear, moment and deflection. Note that the effects of the soil movement extend much farther than the moisture variation distance. The moment and shear distribution close to the edge resemble cantilever behavior.

3.1.2 Moment. The extent of significant moments is illustrated in Figure A1. The length of the equivalent cantilever can be taken as a basic length (Lo) which is dependent on the moisture variation distance, times a correction factor (C) which accounts for secondary effects of several parameters. The value of the correction factor will usually be slightly greater or less than unity. The correction factor was developed to permit accurate approximations of computer results. It was developed from the ratios of actual values to usual values for significant parameters. For example, the "usual" values are: Ym = 1 in, $I = 1500\ in^4/ft$, Pp = 3000 lb/ft. Thus:

$$C = (Ym/1.0)^{.12}\ (I/1500)^{.16}\ (3000/Pp)^{.12}$$

$$C = .8\ Ym^{.12}\ I^{.16} /\ Pp^{.12}$$

A similar approach was used to develop all the formulas in Part II which have an exponential format.

3.1.3 Shear. Maximum shear occurs near the support of the equivalent cantilever. The extent of significant shears is illustrated in Figure A1.

3.1.4 Deflection. Formulas for deflection include an assumed concrete modulus of elasticity Ec = 3,320,000 psi, for both center lift and edge lift.

Vertical movement at the perimeter is much greater than the bending deflection of the equivalent cantilever. To predict the deflection it is necessary to consider translation and rotation at the support of the equivalent beam. The most significant component is due to rotation at the support. These components of deflection are shown in Figure A2. The sum of the cantilever bending and the support translation are approximated by the value 0.11 inch. The percent error due to this approximation is negligible when total deflections are large. The percent error is greater when total deflections are small, but then the deflections are not significant anyway.

Allowable deflections (see Part I, reference 1.1) are expressed as a ratio of the difference in vertical movement at any two points, compared to the distance between those points. For example: $D \leq L/600$, where D is the differential displacement. In such formulas it is appropriate to use the point of maximum deflection and a point of near-zero deflection as the two measuring points. For center lift behavior the maximum deflection occurs at the perimeter, and deflections tend to die out at approximately 4Lc (four times the equivalent cantilever length) from the perimeter. Therefore, the ratio D/4Lc is appropriate for comparison with allowable deflections.

3.2 TRANSVERSE RIB - EDGE LIFT.

3.2.1 General. Typical behavior of a transverse rib for edge lift conditions is shown in Figure A3. This illustrates the soil bearing pressure and the shear, moment and deflection. Soil swell lifts the edge of the ribbed mat, which actually rises off the soil for some distance from the perimeter. For shear and moment, this portion of the rib acts as a simply supported beam spanning between soil support at the perimeter and at an interior location.

3.2.2 Deflection. Vertical movement at the perimeter is driven by the tendency of the soil to swell, and is resisted by the downward loads applied on the soil. As the soil swells at the perimeter the slab is lifted off the interior soil. This concentrates soil reactions near the edge, causing very high pressures. The pressures rise so high that they match the swell pressure of the soil. Thus, the soil cannot swell as much as it would if not loaded. Deflections can be predicted by balancing the upward force of the soil (the swell pressure times the bearing width) with the downward force of applied loads. This downward force can be determined from statics once an equivalent simple beam length is determined. The method for determining the deflection is shown in Figure A4.

Allowable deflections are expressed as ratios, as discussed in the commentary on paragraph 3.1.4. From Figure A3 it can be seen that the appropriate values for this ratio are the edge deflection and the equivalent simple beam length (D/Lc).

Edge lift deflections are mainly a function of soil properties and applied loads, bending stiffness of the ribs has only a secondary effect. Therefore, it may not be possible to control deflections by increasing the rib stiffness. It may be necessary to accommodate calculated deflections by using a less brittle superstructure or by detailing the superstructure to make it less sensitive to deflections. Or it may be necessary to modify soil properties to minimize the edge heave.

3.2.3 Moment. The moments can be calculated by statics, using the equivalent simple beam. The maximum moment will occur at the point of zero shear. Note that the maximum moment is quite sensitive to the beam length, therefore the iterative solution for deflection must converge accurately before calculating moments.

3.2.4 Shear. Shears can also be calculated by statics from the equivalent simple beam. Note that shears will reduce gradually to near-zero around the interior end of the beam because of the distributed soil support.

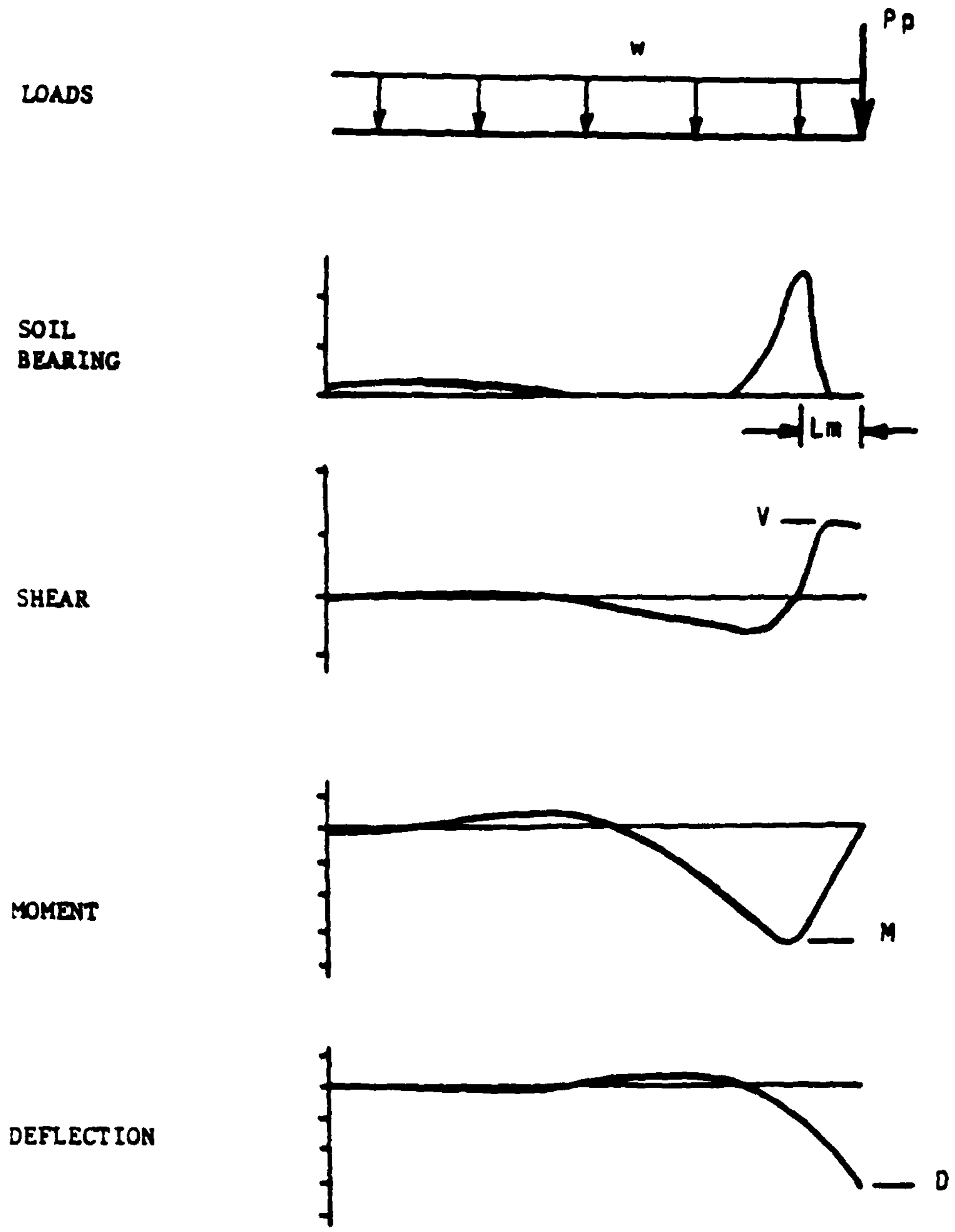
3.2.5 Special Cases. If no concentrated interior load exists, or if it is very far from the perimeter, the formula for the simple beam length must be adjusted as shown. This adjusted formula was also developed to duplicate results from computer solutions.

3.3 PERIMETER RIB.

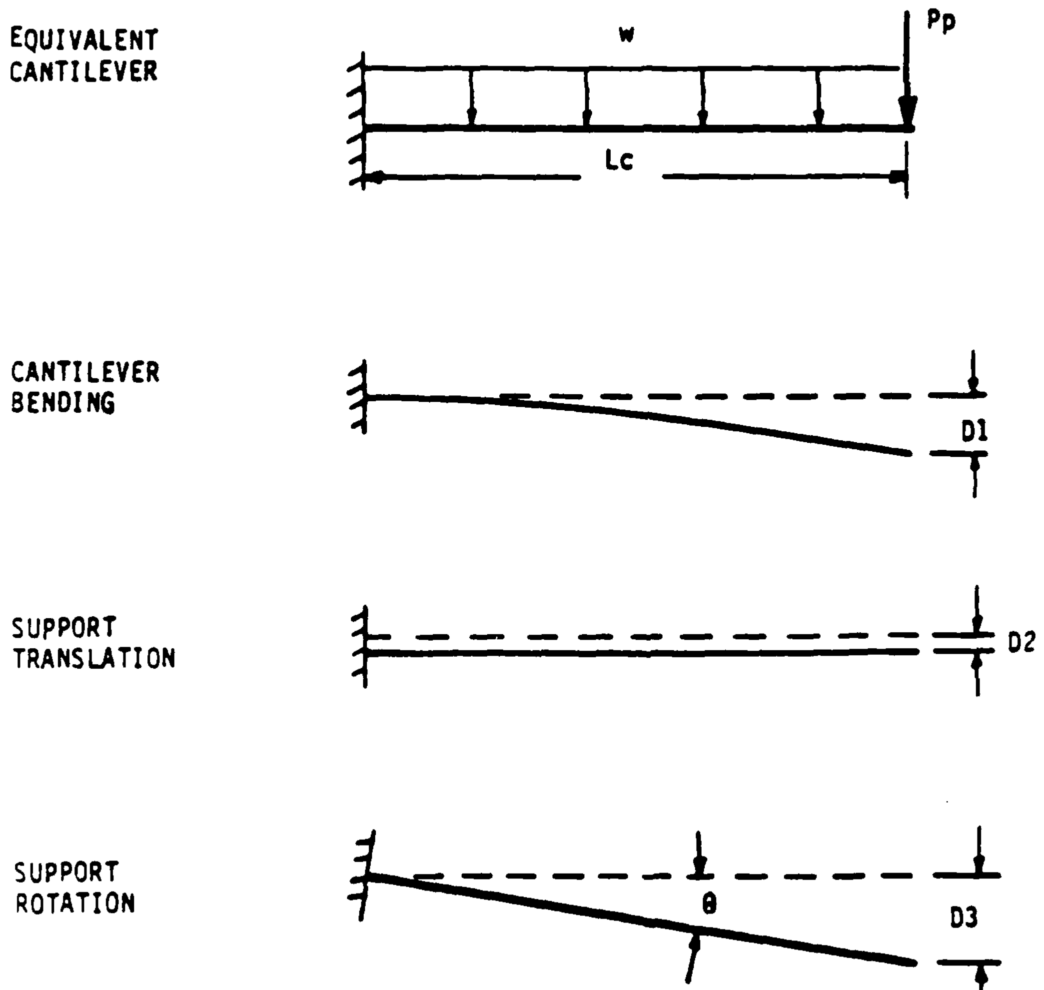
3.4 DIAGONAL RIB.

3.5 INTERIOR RIB. Potential soil heaves in the interior are unpredicatable and are generally due to localized moisture conditions, for example, due to a leaking pipe. Such conditions cannot be accounted for by design forumlas. Adequate strength and stiffness for such unpredictable heaves should be supplied by the minimum requirements listed in Part I of the report. For interior wall or column loads the interior ribs should be designed in accordance with Part I, section 3.2.

FIGURE A1 - CENTER LIFT BEHAVIOR



<u>FIGURE A2 - CENTER LIFT DEFLECTION</u>



D = D1 + D2 + D3

D1 + D2 = .11

D3 = 12 Lc θ

FIGURE A3 - EDGE LIFT BEHAVIOR

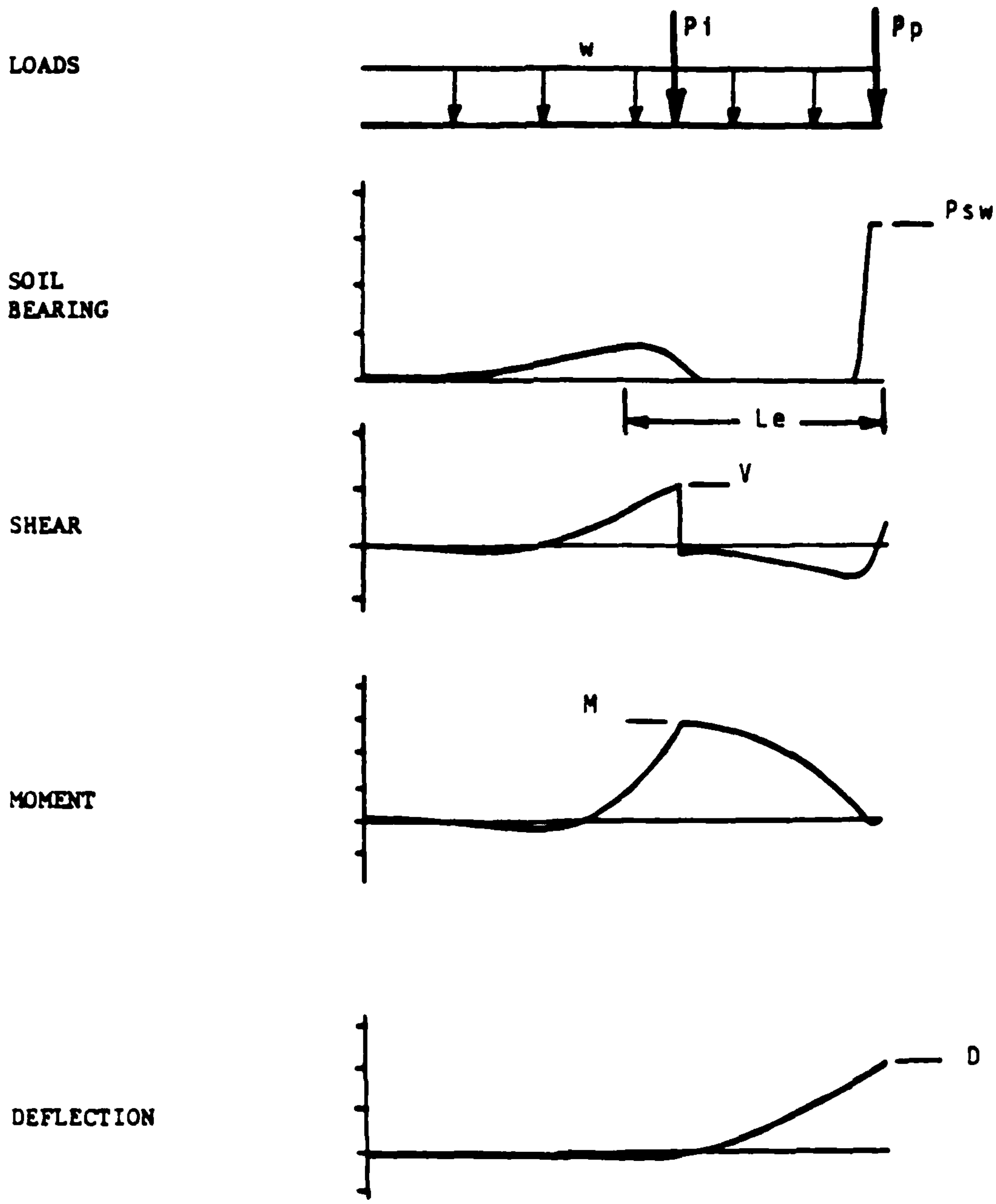
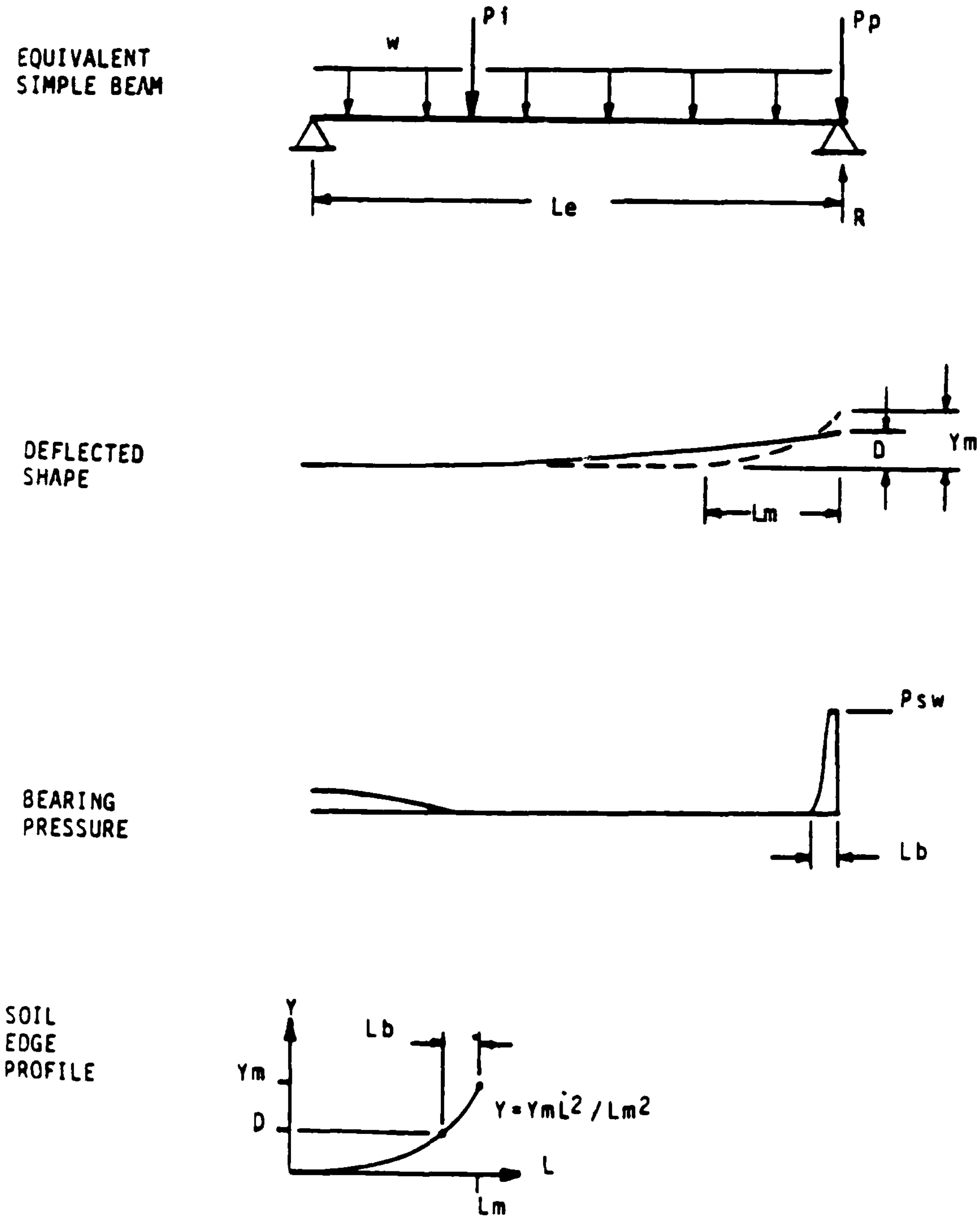


FIGURE A4 - EDGE LIFT DEFLECTION



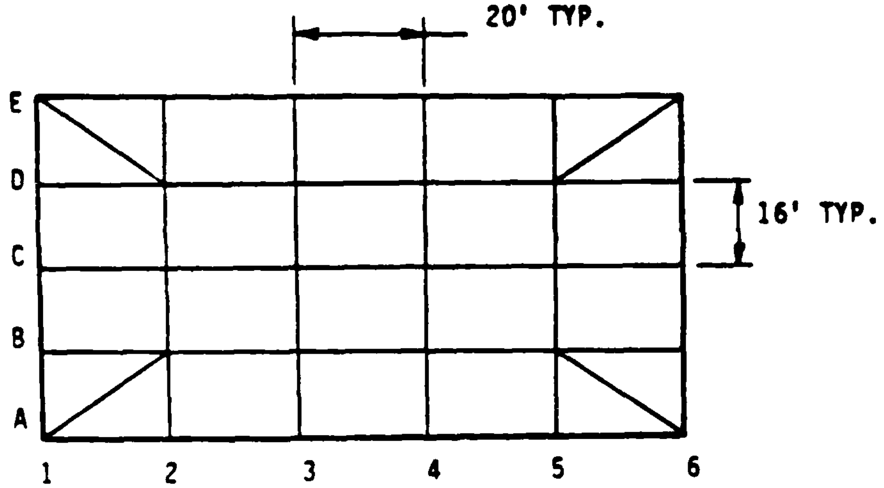
APPENDIX B - DESIGN EXAMPLE

(RIBBED MAT DESIGN IN EXPANSIVE SOIL)

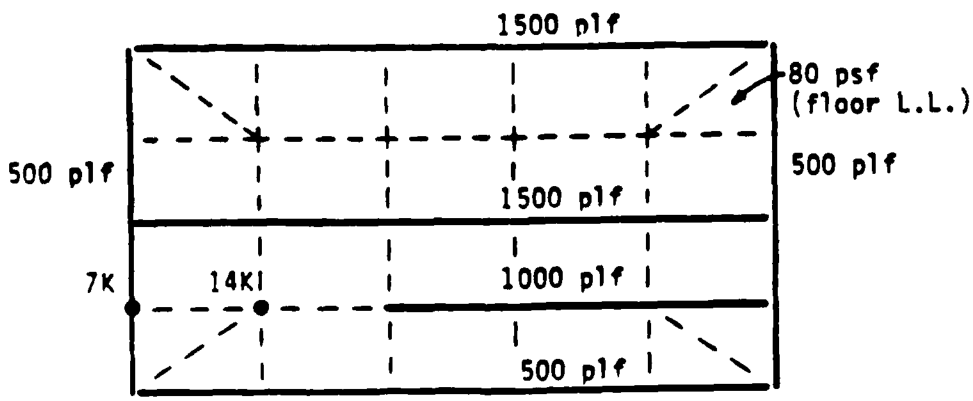
1. SOIL DATA (ref. Part I - 3.3)

qa = 2000 psf
Psw = 3000 psf
k = 100 pci
Lm = 6 ft
Ym = 1.5 in for center lift
Ym = 1.0 in for edge lift

2. FOUNDATION PLAN (ref. Part I - 4.3)



3. LOADS



4. BEARING DESIGN FOR RIBS (ref. Part I - 4.4)

Maximum wall load (P) = 1500 plf

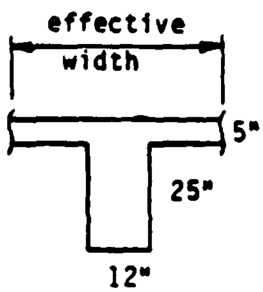
Width $\geq$ P/qa = 1500/2000 = .75 ft

Use 12 inch wide ribs (minimum)

5. INTERIOR RIB PROPERTIES (ref. Appendix A - 2.1)

Ec = 3,320,000 psi

(effective flange width
per ACI 318, section 8.10.2
For "span length" use 4Lc
for center lift or Le for
edge lift)



Let Ir = 36,000 in^4 for center lift
Ir = 24,000 in^4 for edge lift
(ref. ACI 318, section 9.5.2.3, verify Ir after calculating M)

I = Ir/S (in^4/ft):

Rib spacing	16 ft	20 ft
Center lift Edge lift	2250 1500	1800 1200

6. CENTER LIFT DESIGN - RIB E3/C3

6.1 Loads (ref. Appendix A - 2.1)

slab weight = 150 pcf x 5/12 ft = 62 psf

w = DL + LL = 62 + 80 = 142 psf

rib weight = 150 pcf x 2.5 ft x 1.0 ft = 375 plf

Pp = rib + wall = 375 + 1500 = 1875 plf

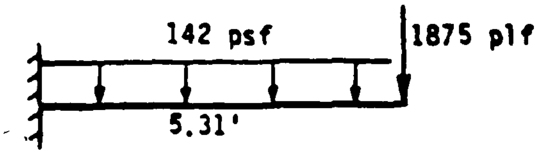
6.2 Equivalent cantilever (ref. Part II - 3.1)

$Lo = 2.3 + .4\ Lm = 2.3 + (.4 \times 6) = 4.7$ ft

$C = .8\ Ym^{.12}\ I^{.16} / Pp^{.12}$

$C = .8 \times 1.5^{.12} \times 1800^{.16} / 1875^{.12} = 1.13$

$Lc = Lo\ C = 4.7 \times 1.13 = 5.31$ ft



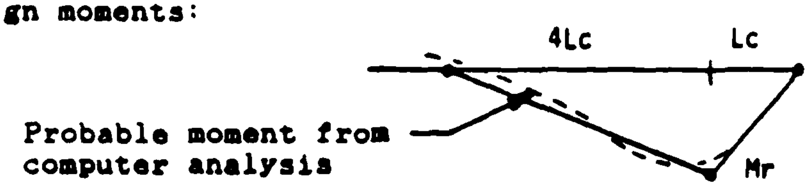
6.3 Moment (ref. Part II - 3.1.2)

$M = Pp\ Lc + 1/2\ w\ Lc^2$

$M = 1875 \times 5.31 + 1/2 \times 142 \times 5.31^2 = 12,000$ ft-lb/ft

$Mr = M \times S = 12000 \times 20 = 240,000$ ft-lb/rib

Design moments:

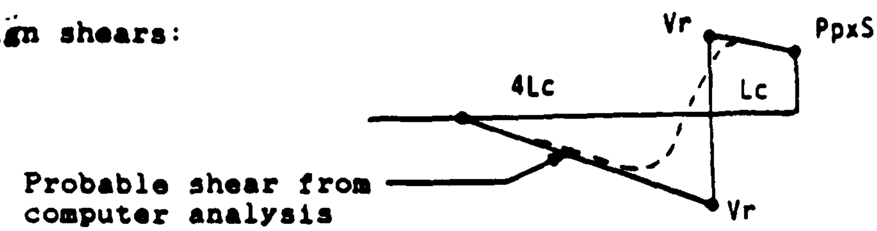


6.4 Shear (ref. Part II - 3.1.3)

$V = Pp + w\ Lc = 1875 + 142 \times 5.31 = 2630$ lb/ft

$Vr = V \times S = 2630 \times 20 = 52,600$ lb/rib

Design shears:



6.5 Reinforcing in rib (ref. Part I - 3.1.6 and 4.5)

$As = (Mr/ad)/1.33$

$As = 240 / (1.76 \times 28 \times 1.33) = 3.66 \text{ in}^2$ (top)

use 3 #10 bars

$v = Vr/bd = 52600 / (12 \times 28) = 157$ psi

$vc = (1.1\sqrt{f'c})1.33 = 80$ psi

$Av = (v-vc)b\ s/(fs\ 1.33)$

$Av = (157-80)\ 12 \times 12 / (24000 \times 1.33) = .35 \text{ in}^2/\text{ft}$

use #4 stirrups @ 12 in

6.6 Deflection (ref. Part II - 3.1.4)

$\theta = M^{1.4} / 9800\ I\ k^{.5}$

$\theta = 12000^{1.4} / (9800 \times 1800 \times 100^{.5}) = .0029$ radians

$D = .11 + 12\ Lc\ \theta = .11 + 12 \times 5.31 \times .0029 = .29$ in

$D/4Lc = .29 / (4 \times 5.31 \times 12) = 1/879$ O.K.

7. EDGE LIFT DESIGN - RIB A2/C2

7.1 Loads

$w = 142$ psf (same as above)

Pp = rib + wall = 375 + 500 = 875 plf

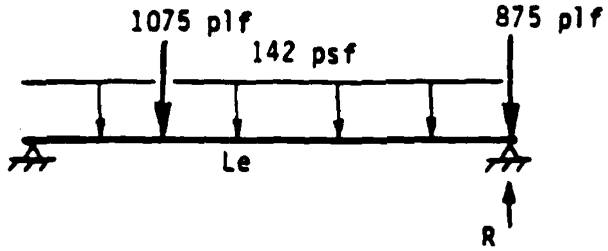
Pi = rib + wall* = 375 + 700 = 1075 plf

* equivalent wall load = column load / rib spacing

14000/20 = 700 plf (ref. Appendix A - 2.4)

$Li = 16$ ft

7.2 Equivalent simple beam (ref. Appendix A - 3.2.1)



7.3 Deflection (ref. Part II - 3.2.2)

$Le = 7.5\ I^{.17}\ Li^{.37}\ D^{.12} / w^{.07}\ Pi^{.11}$

$Le = 7.5 \times 1200^{.17} \times 16^{.37} \times D^{.12} / 142^{.07} \times 1075^{.11}$

$Le = 22.9\ D^{.12}$

assume D = .80 in (slightly less than Ym = 1.0 in)

$Le = 22.9 \times .80^{.12} = 22.3$ ft

$R = Pp + 1/2\ w\ Le + Pi(Le-Li)/Le$

$R = 875 + (142 \times 22.3)/2 + 1075(22.3-16.0)/22.3 = 2762$ plf

$Lb = 1.1(R/Psw) = 1.1(2762/3000) = 1.01$ ft

$D = Ym(Lm-Lb)^2/Lm^2$

$D = 1.0(6.0-1.01)^2/6.0^2 = .69$ in $\neq$ .80 inch assumed!

assume D = .69 in

$Le = 22.9 \times .69^{.12} = 21.9$ ft

$R = Pp + 1/2\ w\ Le + Pi(Le-Li)/Le$

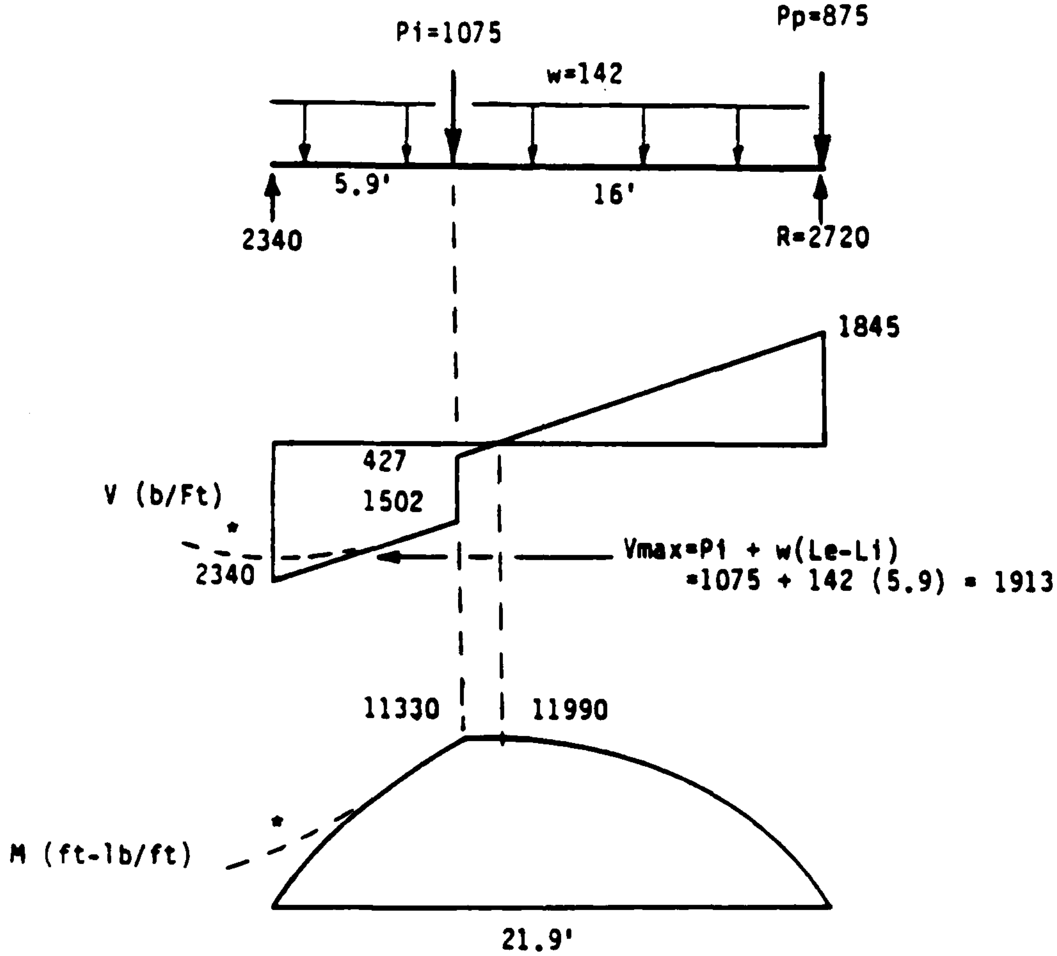
$R = 875 + (142 \times 21.9)/2 + 1075(21.9-16.0)/21.9 = 2720$ plf

$Lb = 1.1(r/Psw) = 1.1(2720/3000) = 1.00$ ft

$D = 1.0(6.0-1.00)^2/6.0^2 = .69$ in CONVERGED!

$D/Le = .69/(21.9 \times 12) = 1/381$ O.K. for non-brittle walls

7.4 Moment and shear (ref. Part II - 3.2.3 and 3.2.4)



* probable shear and moment from computer analysis, note that calculated V=2340 lb will not occur, due to the effects of distributed support from the soil

8. EDGE LIFT DESIGN - RIB E4/C4

8.1 Loads

w = 142 psf (same as above)

Pp = 1875 plf (same as rib E3/C3)

Li = 32 ft (wall along rib C1/C8)

8.2 Deflection

since Li>Le use:

$Le = 10.5\ I^{.17}\ D^{.12} / w^{.07}$ (ref. Part II - 3.2.5)

$Le = 10.5 \times 1200^{.17} \times D^{.12} / 142^{.07} = 24.77\ D^{.12}$

assume D = .62 in

$Le = 24.77 \times .62^{.12} = 23.4$ ft

$R = Pp + 1/2\ w\ Le = 1875 + (142 \times 23.4)/2 = 3536$ plf

$Lb = 1.1(R/Psw) = 1.1(3536/3000) = 1.30$ ft

$D = Ym(Lm-Lb)^2/Lm^2$

$D = 1.0(6.0-1.30)^2/6.0^2 = .614$ inch CONVERGED!

8.3 Find shears and moments by statics, similar to rib A2/C2.

9. CENTER LIFT DESIGN - RIB C1/C3

9.1 Loads

w = slab + LL + wall* = 62 + 80 + 94 = 236 psf

* wall = wall load / rib spacing = 1500/16 = 94 psf
(ref. Appendix A - 2.4)

Pp = rib + wall = 375 + 500 = 875 plf

9.2 Equivalent cantilever

$Lo = 2.3 + .4\ Lm = 2.3 + (.4 \times 6) = 4.7$ ft

$C = .8\ Ym^{.12}\ I^{.16} / Pp^{.12}$

$C = .8 \times 1.5^{.12} \times 2250^{.16} / 875^{.12} = 1.28$

$Lc = Lo\ C = 4.7 \times 1.28 = 6.02$ ft

9.3 Moment

$M = Pp\ Lc + 1/2\ w\ Lc^2$

$M = 875 \times 6.02 + (236 \times 6.02^2)/2 = 9544$ ft-lb/ft

$Mr = M \times S = 9544 \times 16 = 153{,}000$ ft-lb/rib

9.4 Shear

$V = Pp + w\ Lc = 875 + (236 \times 6.02) = 2296$ plf

$Vr = V \times S = 2296 \times 16 = 36{,}700$ lb/rib

9.5 Deflection

$\theta = M^{1.4} / 9800\ I\ k^{.5}$

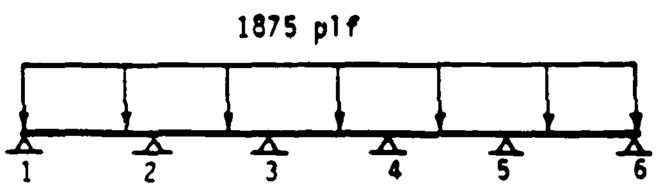
$\theta = 9544^{1.4} / 9800 \times 2250 \times 100^{.5} = .0017$ radian

$D = .11 + 12\ Lc\ \theta = .11 + (12 \times 6.02 \times .0017) = .23$ in

10. CENTER LIFT DESIGN - PERIMETER RIB E1/E6 (ref. Part II - 3.3.1)

10.1 Span between transverse ribs

Pp = 1875 plf (from calculations for rib E3/C3)

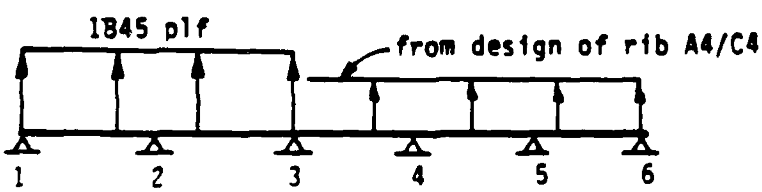


10.2 Analyze by conventional methods

11. EDGE LIFT DESIGN - PERIMETER RIB A1/A3 (ref. Part II - 3.3.2)

11.1 Span between transverse ribs for net upward force (from calculations on rib A2/C2)

R - Pp = 2720 - 875 = 1845 plf (upward)



11.2 Analyze by conventional methods

12. CENTER LIFT DESIGN - DIAGONAL RIB A1/B2 (ref. Part II - 3.4)

12.1 Provide the larger shear and moment capacity of rib B1/B2 or rib A2/B2.

13. RIB D3/D4 (ref. Part I - 4.5)

13.1 Interior rib with no wall or column loads

$As \geq .005\ Ag = .005 \times 12 \times 30 = 1.80\ in^2$ (top and bottom)

This is the typical minimum reinforcement for the full length of all ribs.

APPENDIX B:

PROPOSED RESEARCH PLAN:

PROBLEMS IN NEED OF RESEARCH

by

W. Kent Wray

I. OBJECTIVE(S):

1. DETERMINE THE BEST DESIGN PROCEDURE
2. DETERMINE THE BEST CONSTRUCTION METHOD
3. DEVELOP A NEW DESIGN PROCEDURE
4. TEST THE EXISTING PROCEDURE(S) [VALIDATION/ COMPARISON]
5. DEVELOP AN ANALYSIS MODEL OR MODELS?
6. IMPROVE SITE PREPARATION PROCEDURES [COST CONSIDERATIONS; UTILITIES]
7. REPAIR OF DAMAGED SLABS OR MATS [PERFORMANCE; ECONOMICS]
8. VALIDATE/COMPARE INSTRUMENTATION
9. DETERMINE WHAT SOIL PARAMETERS ARE NEEDED TO PROPERLY DESIGN [INVESTIGATION/CHARACTERIZATION]
10. UNIFICATION OF DESIGN APPROACH
11. BETTER CHARACTERIZATION OF SITE SOIL PROPERTIES
12. "SOIL-STRUCTURE" INTERACTION STUDIES
13. LONG-TERM, COMPREHENSIVE STUDY PROGRAM
14. LESS STRUCTURAL CONSIDERATIONS NOW THAN GEOTECHNICAL CONSIDERATIONS
15. COMPARE METHODS OF PREDICTING SOIL MOVEMENT

IA. CONSOLIDATED OBJECTIVES:

1. BETTER CHARACTERIZATION OF SITE SOIL PROPERTIES
2. SOIL-STRUCTURE INTERACTION STUDIES
3. COMPARE METHODS OF PREDICTING SOIL MOVEMENTS
4. TEST EXISTING PROCEDURES VS. LONG-TERM PERFORMANCE
5. DEVELOP A BETTER ANALYSIS MODEL
6. LONG-TERM COMPREHENSIVE PLAN
7. LESS STRUCTURAL CONSIDERATIONS NOW IN FAVOR OF BETTER UNDERSTANDING OF GEOTECHNICAL PARAMETERS

II. VARIABLES AFFECTING THE S.O.G./S.E. PROBLEM:

CLIMATE

SOIL PROPERTIES

MINERALOGY

CLAY CONTENT

DEPTH OF ACTIVE ZONE

TIME OF CONSTRUCTION

STRATIGRAPHY

SURFACE SLOPE OR TOPOGRAPHY

CHANGE IN SOIL MOISTURE CONTENT

MAT OR SLAB DIMENSIONS

SHAPE OF MAT OR SLAB

STIFFNESS OF MAT OR SLAB

SITE/SUBGRADE PREPARATION

METHOD OF BACKFILL

EXTENT OF CLAY FRACTURE --MACRO/MICRO

BELOW-SLAB UTILITIES

FOUNDATION PREPARATION

PRE-CONSTRUCTION VEGETATION

POST-CONSTRUCTION VEGETATION

LOAD MAGNITUDES

LOCATION OF LOADS

TYPE OF LOADS

METHOD OF MAT OR SLAB REINFORCEMENT

LOCATION OF GROUND WATER TABLE

FIELD PERMEABILITY (FIELD CONDUCTIVITY)

III. RELATIVE IMPORTANCE OF THE VARIABLES:

MOST IMPORTANT	MODERATELY IMPORTANT	LESS IMPORTANT
*CLIMATE	TOPOGRAPHY	TIME OF CONSTRUCTION
SOIL PROPERTIES	MAT/SLAB DIMENSIONS	SHAPE OF MAT
MINERALOGY	BELOW-SLAB UTILITIES	STIFFNESS OF MAT
CLAY CONTENT		SITE PREP.
*DEPTH OF ACTIVE ZONE		METHOD OF BACKFILL
*CHANGE IN SOIL MOISTURE CONTENT		LOAD MAGNITUDE
*EXTENT OF CLAY FRACTURING		LOAD LOCATION
FOUNDATION PREP.		LOAD TYPE
PRE-CONSTRUCTION VEGETATION		METHOD OF REINFORCEMENT
*LOCATION OF GROUND WATER TABLE		
*FIELD PERMEABILITY		

*INTERELATED

USER VARIABLES

POST-CONSTRUCTION VEGETATION

CHANGE IN STRUCTURAL USE

SITE DRAINAGE PATTERN

IV. VARIABLES THAT CAN OR CANNOT BE MODIFIED OR CONTROLLED FOR DESIGN/CONSTRUCTION:

CAN MODIFY OR CONTROL	HOW
SOIL PROPERTIES	CHEMICALLY, MECHANICALLY PRE-WETTING
MINERALOGY	REMOVAL/REPLACEMENT
CLAY CONTENT	REMOVAL/REPLACEMENT
CHANGE IN SOIL MOISTURE CONTENT	COVER SOIL SURFACE; ACTIVE SYSTEM; CONSTRUCTION
FOUNDATION PREPARATION	SPECIFICATION/INSPECTION
LOCATION OF GROUNDWATER TABLE	ARTIFICALLY MODIFY
BELOW-SLAB UTILITIES	SPECIFICATION/INSPECTION
TOPOGRAPHY	SPECIFICATION/INSPECTION

CANNOT MODIFY OR CONTROL	REASON
CLIMATE	NATURAL EVENT
DEPTH OF ACTIVE ZONE	FUNCTION OF NATURAL EVENT
CHANGE IN POST-CONSTRUCTION SOIL MOISTURE CONTENT OR SOIL SUCTION	CANNOT TRULY CONTROL
CLAY FRACTURE	FUNCTION OF NATURAL EVENT
PRE-CONSTRUCTION VEGETATION	NATURAL EVENT
LOCATION OF GROUNDWATER TABLE	NATURAL EVENT
FIELD PERMEABILITY	RANDOM PROPERTY THAT CAN BE MODIFIED BUT NOT CONTROLLED
STRATIGRAPHY	RANDOM EVENT THAT CAN BE MODIFIED BUT NOT CONTROLLED

V. WHAT VARIABLES DO WE WANT TO CONSIDER?

DEPTH OF ACTIVE ZONE

CHANGE IN SOIL MOISTURE CONTENT/CHANGE IN SOIL SUCTION

EXTENT OF CLAY FRACTURING

LOCATION OF GROUNDWATER TABLE

CLIMATE

TOPOGRAPHY

TIME

CLAY MINERALOGY

CLAY CONTENT

STRATIGRAPHY

PRE-CONSTRUCTION VEGETATION

SOIL CHARACTERIZATION PROPERTIES

FOUNDATION PREPARATION

BELOW-SLAB UTILITIES

VI. METHODS OF MEASURING THE SELECTED VARIABLES:

VARIABLE	METHOD	DIRECT/INDIRECT MEASUREMENT	MEASURING CAUSE OR EFFECT
CLIMATE	PRECIPITATION; TEMPERATURE	AT SITE: DIRECT	CAUSE
DEPTH OF ACTIVE ZONE	INCREMENTAL BENCHMARKS; SOIL SUCTION VS. DEPTH	DIRECT --- INDIRECT	EFFECT (OF CLIMATE); CAUSE (OF MAGNITUDE OF MOVEMENT)
CHANGE IN SOIL MOISTURE CONTENT	MOISTURE CELL SOIL SUCTION SOIL MODULUS	INDIRECT INDIRECT INDIRECT	EFFECT (OF CLIMATE; CAUSE (OF MAGNITUDE OF MOVEMENT)
EXTENT OF CLAY FRACTURE	TO BE DETERMINED	TO BE DETERMINED	EFFECT (OF CLIMATE); CAUSE (OF SOIL VOLUME CHANGE)
LOCATION OF GWT	MULTI-LEVEL PIEZOMETERS	DIRECT	CAUSE (OF MAGNITUDE OF MOVEMENT)
FIELD PERMEABILITY	TO BE DETERMINED	INDIRECT (DIRECT BELOW GWT)	CAUSE (OF MAGNITUDE OF MOVEMENT); EFFECT (OF CLIMATE)
TOPOGRAPHY	CONVENTIONAL ENGINEERING SURVEYING PROCEDURES	DIRECT	CAUSE
TIME	CHRONOLOGICALLY	DIRECT	EFFECT

VII: FREQUENCY OF MEASUREMENTS:

MEASUREMENT	FREQUENCY
CLIMATE	DAILY
CHANGE IN SOIL MOISTURE CONTENT	MONTHLY OR EVENT RELATED (DAILY IF AUTOMATED)
CHANGE IN SOIL SUCTION	MONTHLY OR EVENT RELATED (DAILY IF AUTOMATED)
DEPTH OF ACTIVE ZONE	MONTHLY OR EVENT RELATED (DAILY IF AUTOMATED)
EXTENT OF CLAY FRACTURE	MONTHLY OR EVENT RELATED
LOCATION OF GROUNDWATER TABLE	MONTHLY (OR DAILY IF AUTOMATED)
FIELD PERMEABILITY	ONCE EACH SEASON IN FIRST YEAR; ANNUALLY THERE-AFTER; ONCE EACH SEASON LAST YEAR OF STUDY
TOPOGRAPHY	ONCE
SITE SURFACE ELEVATION CHANGES	MONTHLY; QUARTERLY

VIII. DURATION OF TEST:

MINIMUM DURATION: THREE YEARS

PREFERRED DURATION; TEN YEARS

IX. LOCATION(S) OF TEST SITE(S)

PREFER TEST SITE TO BE ON PROPERTY OWNED BY RESEARCH SPONSOR TO ENSURE CONTINUITY

PREFER TEST SITE TO LOCATED NEAR PERSONNEL WHO WILL BE MAKING FIELD MEASUREMENTS

SOME SUGGESTIONS:

FORT SAM HOUSTON, SAN ANTONIO, TX
(DRY CLIMATE)

LACKLAND AFB, SAN ANTONIO, TX
(DRY CLIMATE)

RED RIVE ARMY DEPOT, TEXARKANA, ARK
(WET CLIMATE)

X. ANCILLARY STUDIES:

STUDY	PURPOSE
EXTENT OF CLAY FRACTURE/FRACTURE MECHANICS	EFFECT OF CRACKING ON MAGNITUDE OF SOIL VOLUME CHANGE TRANSMITTED TO MAT/SLAB FOUNDATION
SOIL MOISTURE DIFFUSIVITY	ASSOCIATED WITH VELOCITY WITH WHICH MOISTURE CHANGES OCCUR IN THE SOIL MASS
ANISOTROPY	INFLUENCES DIRECTION, RATE, AND AMOUNT OF MOISTURE WHICH MOVES THROUGH SOIL MASS
MODULUS OF SUBGRADE REACTION/MODULUS OF ELASTICITY	ANALYSIS MODELS REQUIRE SUBGRADE REACTION OR ELASTICITY MODULUS TO REPRESENT SOIL SUPPORTING MAT OR SLAB; REALISTIC REPRESENTATION OF SOIL RESPONSE INCREASES VALIDITY OF ANALYSIS

APPENDIX C:

A PROPOSED PLAN OF STUDY

by

W. Kent Wray

I. OBJECTIVE:

ASSUMPTION 1: QUANTIFIABLE EFFECTS OF SITE SOIL ON SLAB PERFORMANCE IS OF MORE IMMEDIATE CONCERN.

ASSUMPTION 2: STRUCTURAL EFFECTS OF SLAB RESPONSE TO EXPANSIVE SOIL MOVEMENT IS OF LESSER CONCERN INITIALLY.

THEREFORE: INITIAL OBJECTIVE IS TO STUDY SITE SOIL VARIABLES WITH AN OVERALL GOAL OF DEVELOPING RATIONAL METHODS OF INCORPORATING CURRENTLY UNQUANTIFIABLE (OR POORLY QUANTIFIABLE) VARIABLES INTO MAT FOUNDATION DESIGN AND/OR CONSTRUCTION METHODS.

ULTIMATELY: PREDICT MAXIMUM EXPECTED DIFFERENTIAL SOIL MOVEMENT AND EDGE MOISTURE VARIATION DISTANCE FOR:

-- ANALYSIS (SOIL-STRUCTURE INTERACTION)

-- STRUCTURAL DESIGN

II. MAJOR TASKS AND SUBTASKS OF PROPOSED STUDY:

A. ACTIVE ZONE DEPTH, Z_A:

- -- ADOPT AN ACCEPTABLE DEFINITION
- -- PERIODICALLY MEASURE A_Z
- -- DEVELOP METHOD OF PREDICTING OR ESTIMATING A DESIGN A_Z

B. SURFACE CRACKING:

- -- PERIODICALLY EVALUATE OR MEASURE SURFACE CRACKING PATTERN
- -- PERIODICALLY MEASURE CRACK WIDTH/DEPTH
- -- PERIODICALLY MEASURE CHANGE IN SURFACE ELEVATION OF CRACKED SURFACE
- -- PERIODICALLY MEASURE CHANGE IN SOIL MOISTURE CONTENT AND/OR SOIL SUCTION
- -- DEVELOP METHOD OF PREDICTING EFFECT OF CRACKING ON SOIL VOLUME CHANGE (SURFACE SHRINK/HEAVE)

C. MOISTURE MOVEMENT INTO AND OUT OF SOIL:

- -- MEASURE SOIL DIFFUSIVITY
- -- MEASURE SOIL MASS PERMEABILITY (CONDUCTIVITY)
- -- MEASURE EVAPORATION
- -- MEASURE PLANT TRANSPIRATION
- -- DEVELOP METHOD OF ESTIMATING RATE OF MOISTURE MOVEMENT IN SOIL

D. RELATE CLIMATE TO CLIMATE-DEPENDENT VARIABLES:

- -- MEASURE SITE TEMPERATURE AND PRECIPITATION
- -- QUANTIFIY CLIMATE AS A RATIONAL CLIMATOLOGICAL INDEX (E.G., THORNTHWAITE MOISTURE INDEX)

II. MAJOR TASKS AND SUBTASKS (CONT'D):

E. CHARACTERIZE TEST SITE(S):
- -- GEOLOGY
- -- STRATIGRAPHY
- -- INDIVIDUAL SOIL STRATA PROPERTIES
- -- TOPOGRAPHY
- -- VEGETATION
- -- GROUNDWATER TABLE
- -- CLIMATE

F. SOIL RESPONSE TO NATURAL INFLUENCES WITH TIME:
- -- CHANGE IN SURFACE ELEVATION
- -- CHANGE IN SOIL MOISTURE CONTENT
- -- CHANGE IN SOIL SUCTION

G. SOIL MODULI:
- -- EVALUATE MODULUS OF SUBGRADE REACTION, K
- -- EVALUATE MODULUS OF ELASTICITY, E_S

H. FIELD INSTRUMENTATION:
- -- EVALUATE, SELECT, CONSTRUCT, CALIBRATE, INSTALL TO MAKE THESE MEASUREMENTS:
  - -- SOIL MOISTURE CONTENT
  - -- SOIL SUCTION
  - -- BENCH MARKS OF VARYING DEPTH
  - -- WEATHER
  - -- SURFACE ELEVATION POINTS
  - -- SUBGRADE REACTION/ELASTICITY MODULI
  - -- SLAB STRUCTURAL INSTRUMENTATION, E.G., STRAIN GAGES
  - -- SOIL TEMPERATURE
- -- EVALUATE MEASUREMENTS

II. MAJOR TASKS AND SUBTASKS (CONT'D)

I. DEVELOP DESIGN RELATIONSHIP BETWEEN THE SITE SOIL PARAMETERS:

-- ACTIVE ZONE DEPTH, Z_A

-- SURFACE CRACKING, C_R

-- MOISTURE MOVEMENT, V

-- CLIMATE, C_L

-- SITE SOIL CHARACTERIZATION, C_H

$$F = f(Z_A, C_R, V, C_L, C_H)$$

J. RELATE SOIL PARAMETER FACTOR TO PERIMETER MOVEMENT CONDITIONS: E_M AND Y_M.

III. TEST SITE(S) INSTALLATION:

A. SELECT SITE(S):

-- MULTIPLE SITES BEST (E.G., FIVE: DRY, MODERATELY DRY, NEUTRAL, MODERATELY WET, WET)

-- LOCATION/PROPERTY OWNER

B. CHARACTERIZE SITE(S):

-- FIELD

-- LABORATORY

C. CONSTRUCT SITE(S)

-- FLEXIBLE COVER

-- RIGID COVER

-- RESIDENTIAL/LIGHT COMMERCIAL SIZE

-- INDUSTRIAL SIZE

-- CONSTRUCTION TECHNIQUE

-- "CONTROL" MAT OR SLAB

D. INSTALL INSTRUMENTATION

-- INSTALL INSTRUMENTS

-- INSTALL DATA ACQUISITION SYSTEMS

-- ESTABLISH MONITORING OR MEASUREMENT SCHEDULES

IV. MEASUREMENT AND EVALUATION OF IN-PROGRESS PERFORMANCE:

A. DURATION OF TEST PROGRAM AND FIELD MEASUREMENTS:

-- PROPERTY OWNER

-- LIFE OF INSTRUMENTATION

B. METHOD OF MEASUREMENT:

-- AUTOMATIC

-- SEMI-AUTOMATIC

-- MANUAL

C. FREQUENCY OF MEASUREMENTS

-- DAILY

-- MONTHLY

-- BI-MONTHLY

-- QUARTERLY

-- SEASONALLY

-- EVENT-RELATED

V. DEVELOP A DESIGN SITE EVALUATION METHOD FOR MAT/SLAB STRUCTURAL ANALYSIS AND/OR DESIGN:

A. METHODS OF INCORPORATING SITE SOIL PROPERTIES INTO DESIGN METHOD:

- -- ACTIVE ZONE DEPTH
- -- SURFACE FRACTURING
- -- MOISTURE CHANGE OF SOIL
- -- CLIMATE
- -- RATE OF MOISTURE CHANGE IN SOIL
- -- SITE SOIL PROPERTIES

B. METHODS OF EVALUATING OR MEASURING "NON-CONVENTIONAL" SITE SOIL PROPERTIES FOR DESIGN OR ANALYSIS.

C. DEVELOP A METHOD OF ANALYSIS THAT IS CAPABLE OF EMPLOYING THESE HERETOFORE "UNQUANTIFIABLE" SOIL PROPERTIES IN THE ANALYSIS MODEL

D. USE RESULTS OF ANALYSIS IN ONE OF SEVERAL "RATIONAL" DESIGN PROCEDURES TO PRODUCE STRUCTURAL MAT OR SLAB DESIGN.

VI. TEST DEVELOPED METHOD:

A. DESIGN ONE OR MORE MATS BASED ON DEVELOPED METHOD

B. INSTALL APPROPRIATE INSTRUMENTATION TO MONITOR MAT PERFORMANCE

C. EVALUATE PERFORMANCE

D. MODIFY METHOD AS NECESSARY.

APPENDIX D:

WORKING GROUPS

by

W. Kent Wray

POSSIBLE WORKING GROUP TOPICS

by

W. Kent Wray

1. IS THERE A SLAB-ON-GROUND-OVER-EXPANSIVE-SOIL PROBLEM? IF SO, SHOULD RESOLUTION OF THE PROBLEM BE UNDERTAKEN BY INDIVIDUAL RESEARCHERS, PRACTITIONERS, OR AGENCIES AS IS PRESENTLY DONE OR SHOULD THERE BE A UNIFIED APPROACH? LIST SOME ADVANTAGES/DISADVANTAGES OF EACH APPROACH.

2. ARE THERE ADEQUATE SLAB ON GROUND-OVER-EXPANSIVE-SOIL DESIGN METHODS OR PROCEDURES CURRENTLY AVAILABLE? IF SO, WHAT ARE THEIR SHORTCOMINGS? WHAT IS NEEDED TO IMPROVE THEM? WHY ARE THEY/ARE THEY NOT IN GENERAL USE? IS AN ENTIRELY NEW DESIGN PROCEDURE NEEDED? IF SO, HOW WOULD IT BE AN IMPROVEMENT OVER EXISTING PROCEDURES? HOW CAN IT BE ENSURED THAT THIS NEW PROCEDURE WOULD BE MORE WIDELY OR FREQUENTLY USED THAN EXISTING PROCEDURES?

3. SHOULD MORE REGULATORY ACTIONS BE IMPLEMENTED WITH RESPECT TO THIS SLAB-ON-GROUND-OVER-EXPANSIVE-SOIL PROBLEM FROM THE DESIGN VIEWPOINT? FOR EXAMPLE:

   A. MORE RESTRICTIVE BUILDING CODES?

   B. FEDERAL AGENCIES BE MORE RESTRICTIVE ABOUT "ACCEPTABLE" DESIGNS?

   C. REQUIRE FOUNDATION STRUCTURES TO BE CONSTRUCTED IN EXPANSIVE CLAY SOILS TO BE DESIGNED BY "CERTIFIED EXPANSIVE SOIL ENGINEERS" COMPARABLE TO CERTIFIED TAX LAWYERS, CERTIFIED DEFENSE LAWYERS

4. SHOULD MORE REGULATORY ACTIONS BE IMPLEMENTED WITH RESPECT TO THIS SLAB-ON-GROUND-OVER-EXPANSIVE-SOIL PROBLEM FROM THE CONSTRUCTION VIEWPOINT? FOR EXAMPLE:

   A. REQUIRE AT LEAST ONE 15-FT DEEP BORING ON EACH CONSTRUCTION SITE WITH SOME MINIMAL TESTING (E.G., ATTERBERG LIMITS, HYDROMETER, IN SITU SOIL MOISTURE CONTENT, IN SITU SOIL SUCTION BY FILTER PAPER METHOD, ETC) WITH SOME CERTIFICATION BY TESTING LAB REGARDING EXPANSIVE POTENTIAL OF SITE SOIL?

   B. REQUIRE A P.E. SEAL ON ALL FOUNDATION PLANS TO BE CONSTRUCTED IN EXPANSIVE SOIL?

   C. REQUIRE A "CERTIFIED EXPANSIVE SOILS ENGINEER" TO CERTIFY AND SEAL (P.E.) ALL FOUNDATION PLANS TO BE CONSTRUCTED IN EXPANSIVE SOIL?

   D. REQUIRE THE ENGINEER OR DESIGN PROFESSIONAL TO INSPECT CONSTRUCTION ONE OR MORE TIMES DURING CONSTRUCTION PROCESS (E.G., DURING UTILITY INSTALLATION, PRIOR TO CONCRETE PLACEMENT, DRAINAGE/LANDSCAPING NEAR END OF CONSTRUCTION, ETC.

5. IS THERE BENEFIT TO ESTABLISHING A NATIONAL "CENTER" OR "CLEARING HOUSE" WITH A RESPONSIBILITY TO PROVIDE:

A. GUIDANCE INFORMATION TO DESIGN PROFESSIONAL

B. GUIDANCE INFORMATION TO BUILDERS?

C. FREQUENT SHORT COURSES ON THE PROBLEM TO DESIGN PROFESSIONALS, BUILDERS, INSPECTORS, CODE ENFORCERS, ETC?

6. ASSUMING THAT THERE IS A RECOGNIZED PROBLEM AND FURTHER ASSUMING THAT THERE IS A NEED FOR FURTHER RESEARCH TO MITIGATE THE PROBLEM, HOW SHOULD THIS RESEARCH BE ACCOMPLISHED?

A. THROUGH AN EXISTING FEDERAL GOVERNMENT AGENCY

(1) E.G., NSF--PRIMARILY BY UNIVERSITIES

(2) E.G., FEMA, COE, ETC.--PRIMARILY BY GOVERNMENT SCIENTISTS/ENGINEERS WITH SOME UNIVERSITY CONTRIBUTIONS?

B. ESTABLISH A NEW FEDERAL AGENCY/OFFICE THAT ACCOMPLISHES RESEARCH BY EITHER METHOD 6.A(1) OR 6.A(2)?

C. ESTABLISH A UNIFIED ORGANIZATION IN CONJUNCTION WITH AN EXISTING NOT-FOR-PROFIT INSTITUTION (E.G., A UNIVERSITY RESEARCH FOUNDATION) WHICH RECEIVES FUNDING FROM A NUMBER OF PRIVATE AND GOVERNMENT AGENCIES, DETERMINES THE LONG-TERM RESEARCH OBJECTIVES AND PRIORITIES, ALLOCATES RESEARCH FUNDING TO RESEARCH ORGANIZATIONS OR INDIVIDUALS JUDGED TO HAVE THE REQUIRED EXPERTISE/CAPABILITY, AND EVALUATES THE PROGRESS BEING MADE ON THE OVERALL PROBLEM?

7. ASSUMING THAT ADDITIONAL RESEARCH IS NEEDED, HOW SHOULD THIS RESEARCH BE CONDUCTED AND/OR REPORTED SO THAT ITS VALIDITY IS OBVIOUSLY RECOGNIZED AND ITS APPLICATION IS WIDELY ACCEPTED? CONSIDER LABORATORY RESEARCH, NUMERICAL METHODS/MODELING AND THEORETICAL RESEARCH, AND FIELD OBSERVATION/TESTING RESEARCH, WHAT IS THE SINGLE-MOST NEEDED ASPECT OF THE SLAB-ON-GROUND-OVER-EXPANSIVE-SOIL PROBLEM THAT NEEDS TO BE RESEARCHED (I.E., WHERE DO WE BEGIN)?

ANSWERS FROM WORKING GROUPS

Topic 1. This topic was answered yes during the workshop. Resolution of the expansive soil problem must necessarily be done, as a practical matter, by practitioners or agencies as presently done; there should be an improved focus and liaison among principal investigators in government, academia, and commercial organizations. Mr. Robert Crisp indicated that there is no common "best" solution because there is no common problem. Some of the many factors found in practice associated with the overall problem are the degree of expansive characteristics, local climate, economics, constructability, projected beneficial life, and available repair processes. Solutions of these problems should also include real estate developers, contractor organizations, casualty insurance writers and others who can benefit financially from this program. There should be a common clearing house of information to bring all improvements together and disseminate results in a timely and useable manner.

Topic 2 (Hilmer, Lytton, Deddons). Adequate slab design procedures are not available, but good suggestions exist on how to put one together. Drainage and moisture barriers need to be considered as part of the overall design. Need to minimize deformation as much as possible, subject to economic trade-off analysis of first cost versus repair cost. Mr. Crisp indicated that there may be adequate methods available throughout the construction industry, but not disseminated to various agencies, designers, and regions of the country. This question cannot be resolved until all the experience and procedures are pulled together and evaluated from the design, constructability and economic aspects. Any "new" design procedure that is easier to construct and more economical than those being used at present is beneficial.

a. None are based on anisotropic spectra analyses. Deflection versus wave length spectrum criteria for acceptable slab performance do not exist. Good ways have not been developed for predicting the change of spectra with time at the edge and center of an extended area or large mats.

b. Improvements are needed to meet the anisotropic spectra analyses. Distress criteria (e.g., photos showing acceptable and unacceptable ratings) need to be developed to use in establishing acceptable deflection versus wave length criteria. Field observations of the surface movement spectra in and outside of buildings are required to predict the change of spectra with time.

c. New methods (PTI, frequency spectrum) not in general use because little incentive. Regulatory codes and practices do not exist to impose use of any method on design engineers.

d. Entirely new design procedure (in concept) needed for large mat foundations.

e. Requlations and codes are the only thing that can impose wide spread use of any procedure. This is not within the scope of research. It must be promoted by practioners through professional and technical societies, trade organizations, legislatures, etc.

Topic 3 (Jones, Clough, Gompers, Schaefer). Implementation of regulatory actions are not practical. Engineers are outvoted. Code people are mindful of public demand. Federal agencies do not have the manpower to enforce codes. It is not practical to certify expansive soil engineers. Things of value include education by films, educational material to the public, and publicity. Information developed under Corps of Engineer (CE) research programs and training used in CE courses need to be available through technology transfer with academia, other agencies, and commercial organizations. Mr. Crisp indicated he has grave reservations about the effectiveness of regulations that are not "policed" throughout the industry. Regulations regarding "life-safety" and economic (monetary) risks should be differentiated.

Topic 4 (Prager, Blacklock, McKeen). The general answer to this question is no. A "Certified Expansive Soils Engineer" should not be required. Inspection during construction is necessary. Mr. Crisp indicated that the approaches listed are good and desirable, but the problem is implementation, policing and punitive consequences imposed for non-compliance.

Topic 5 (Wray, Fletcher, McAnear). A benefit for establishing a national "mini-center" or University as a point of contact is to get information out. A series of short courses are needed to educate and train professionals. The CE will continue its courses to educate Corps personnel and personnel of other agencies who send representatives to these courses. A general research program is worthwhile to set the right directions, but each agency will pursue its own interests. A strong liaison and communication system is required. The biggest effort will be to sell the program; those most interested in this work will be those who hurt the most. There is a need for multiple entities striving to meet the special needs of designers, construction agencies, contractors, inspectors, code enforcers, etc. Improved communication and liaison is greatly needed. Mr. Crisp indicated that this approach has the greatest possibility of immediate and long-term benefits. WES is the logical choice for such a "clearing house" or center and a source of expertise, either in-house or available under some other device.

Topic 6 (Wray, Fletcher, McAnear). The research work should not be accomplished through a new government agency. A single program through an existing agency or consortium is not practical due to diverse interests and special needs; however, greater communication and liaison between agencies is highly desirable. Cooperation with academia is highly beneficial. Professional liaisons with commercial organizations and private practitioners is desirable. The problems and solutions to problems with foundations on expansive clay should be taught to undergraduate and graduate students and published in proceedings of conferences. Mr. Crisp indicated that topic 6A(2) is the best approach with WES designated as that agency. He also recommends input from other than pure research organizations to include construction organizations, developers, and those with large financial commitments in this area. These are the primary beneficiaries of improvements and would most likely implement changes if there is a recognized benefit to them.

Topic 7 (Stroman, Yunker, Branch, Johnson). Coordinated field and laboratory studies should be conducted in which full communication is required between participants. For supporting the Army, research should be coordinated through WES as an established clearing house. Results should be distributed through WES reports, American Society of Civil Engineer and American Society for Testing and Material publications, and proceedings of specialty conferences. Work should eventually be accepted if it works and it is economical to use. Aspects to be researched are field studies first and laboratory studies second; these go together. Initial needs include measurements for frequency spectrum analysis and record of damages. A systematic damage reporting system that considers repairs should be developed. Data that is collected must be applicable to improvements in available theories. Mr. Crisp believes that the single-most needed aspect is to collect from throughout the building industry a detailed list of all procedures that have been utilized (not just recently), evaluate the end-product (track record), realistically cost-out the procedures recognizing the effect of labor intensive methods, constructability for large and small projects, and risks associated with each and then evaluate future needs.